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CONCRETING THE POWER HOUSE SUBSTRUCTURE AT CHICKAMAUGA DAM
Time Studies Have Effectively Cut the Cost of This and Many Other Operations on TVA Projects (See Page 281)

Volume 9 — AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852 — Number 5

MAY 1939

Among Our Writers

William E. Wickenden has been identified with engineering education at Rochester Mechanics Institute, University of Wisconsin, Massachusetts Institute of Technology, for the Western Electric Company, the War Department, the American Telephone and Telegraph Company, and the Society for the Promotion of Engineering Education. He is now president of the Case School of Applied Science.

T. J. Mitchell (University of Michigan, 1912) was with the U. S. Lake Survey at Detroit until 1920, when he was made assistant professor of geodesy and surveying at his alma mater. In 1934 he became an assistant engineer with the U. S. Coast and Geodetic Survey.

Frank C. Wardwell (Ohio State University, 1911) was in railroad work for several years—location, construction, valuation, and maintenance. More recently he has been cost engineer on several large hydroelectric developments, including Mitchell Dam, Bartletts Ferry, Conowingo, Fifteen Miles Fall Development, McIndoes Falls Development, and Tygart Valley Dam. He has been with the TVA since September 1935.

William H. Trinkaus (Armour Institute of Technology, 1908) entered the service of the Sanitary District of Chicago as a rodman in 1909, and rose through the ranks to become acting chief engineer in 1935 and chief engineer in 1937. In his spare time he studied law, and was graduated from Chicago Kent College of Law in 1917.

Seward H. Mott has had wide experience in the design and development of subdivisions, as a member since 1912 of the firm of Pitkin and Mott of Cleveland, Ohio. For the last 3 years, he has had direct charge of all land planning and city planning activities of the Federal Housing Administration.

Bruce Johnston (University of Illinois, 1930) holds advanced degrees from Lehigh and Columbia universities. Prior to 1932 he had 4 years of varied experience in design and construction and since this time has been active in structural research and teaching. He went to Lehigh in 1938 after four years as instructor at Columbia University.

D. Grant Mickle received a master's degree in street-traffic research at Harvard University (1931). The greater part of his work has been with Jensen, Bowen and Farrell, as manager of that firm's traffic and transport department. Since August 1937 he has also been assistant director and planning engineer for the Michigan Highway Planning Survey.

Harold A. Thomas (Columbia University, 1908) has been professor of hydraulic engineering at Carnegie Institute of Technology since 1923. During intervals in a teaching career he has served as engineer with the Miami Conservancy District, City of Pittsburgh, Flood Commission of Pittsburgh, and the U. S. War Department.

Wallis S. Hamilton (Carnegie Institute of Technology, 1935) has been engaged since 1937 in hydraulic model studies conducted at Carnegie by Professor Thomas. Prior to 1937 he was in the employ of the Tennessee Valley Authority and the U. S. Geological Survey.

Edwin A. Fisher, who was born in Royalston, Mass., July 17, 1847, has an engineering experience of 68 years—16 years on railroads, 40 years on municipal public works, and 12 years (since his "retirement") on studies and reports on water supply, city planning, and flood protection.

P. A. Covas (Cornell University, 1925) was employed by the City of Rochester, Department of Engineering, in 1925 on subway construction. He has been associated with E. A. Fisher on flood protection and other studies since 1928—chiefly on hydraulic research and design.

James B. Hays (University of Idaho, 1911) after 15 years on hydraulic projects in the Far West, entered the employ of the Aluminum Company of America in 1926. His assignments there included resident engineership of Santeetlah and Calderwood dams. From 1934 until 1936, when he assumed his present duties with TVA, he was with the U. S. Bureau of Reclamation, specializing in grouting work.

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Something to Think About

A Series of Reflective Comments Sponsored by the Committee on Publications

The Young Engineer Facing Tomorrow

Recent Address to Student Guests of St. Louis Local Section, Summarized for the Benefit of All Graduating Civil Engineers

By William E. Wickenden
President, Case School of Applied Science, Cleveland, Ohio

To any of our older engineers who may have grown pessimistic over the future, an opportunity to rub elbows with you men yet in college ought to be a reassuring experience. You are facing life in an exciting period when change is the only certainty that can be counted on. As engineers you believe in change but distrust propaganda. You realize that the prevailing social scheme is not the product of design, but rather the residue of millions of trial-and-error experiments spread over ten thousand years of civilized history. What worked was retained; what failed was discarded. There is a time lag in this rule, but it works inexorably. When a Karl Marx or an Adolf Hitler goes into the silence and brings forth the blueprint of a new society, you view it with a healthy skepticism. The odds of experience are against it.

Current Experimentation.~As engineers, however, we cannot remain satisfied with progress through trial and error. Our job calls on us to replace guesswork with rational planning, wherever possible. We believe in experimentation, imaginatively conceived, rationally controlled, and rigorously checked. Our interest is roused by two decades of experiment with collectivism in Russia, in Italy, in Germany, and under less coercive guises in the United States. A critical stage in these experiments seems near at hand. The result? The pendulum seems to be starting its long swing back to the ideals of freedom. If that is true, it may mean much to you.

Some fruits of recent experimentation will doubtless endure. Bankers now accept the SEC instead of fighting it. Insurance men are reconciled to social security measures. The utility interests expect government to develop water power. Industry generally is lining up for collective bargaining. Sober citizens who begrudge the waste of public funds on hastily improvised projects recognize the necessity for long-planned spending on public works as a counterpoise against violent swings of the economic cycle. Collective wealth is increasing faster than private wealth. Permanent gains are being made in conservation of soil, water, and mineral resources and in public facilities for education, health, and recreation.

To Make Government Service Better.~As civil engineers most of you may have more to do with public welfare than with private enterprise, and these new trends may largely shape your careers. A growing proportion of you will probably find careers in government service. This may be good for your profession, by making a more even balance between the individualistic practitioner and the career man and by affording a wider choice between technical and executive types of responsibility. Equally, it should be good for government, helping to even the balance between social visionaries and fact-minded men.

If engineers are to take a larger part in government, it is the concern of all of us to make government a better place for engineers to work. We ought to pull together for fair compensation, for a more rational grouping of engineering functions, and for their removal from direct political pressure. This is not a remote issue; it may touch next year's bread and butter for many of you seniors. Honest engineering calls for clearly defined responsibility, for reasonable freedom of initiative, and for non-political tenure. Do you want your job to depend on getting the initials of the right political boss? Do you want your decisions revised in the interest of the election returns? Do you want to be caught in a mesh of red tape? Can you expect to work well in a confused, overlapping, or isolated organization?

Recognizing Economic Facts.~Facing tomorrow, it is time now to quit taking the economic world for granted. It will be well for you to reckon your stake in free institutions and in free enterprise. This principle of enterprise is not something remote or abstruse. It is the inner drive which urges men to get on and not merely to hold on, to depend on their own efforts rather than the paternalism of the state. It spurs the scientist to wrestle with nature, the inventor to strive for a novel product or a new way of doing work, the thrifty man to sacrifice in order to save and to own, the financier to take the risks of a new

industry, the engineer to plan, and the executive to organize for increased efficiency.

What we call capitalism is the fruit of freedom and enterprise in the world of work. Free enterprise, it is pretty generally agreed, will work well only in an expanding economy. In the past, spreading frontiers, virgin resources, and rising population have supplied the expansive force. Now we have to seek it in raising the general standard of living of a population predominantly urban and industrial.

Engineers Prosper with Industry.—Free enterprise, when wisely directed, tries to raise living standards by multiplying wealth, while state paternalism almost always ends up in attempting a solution by dividing it. Every engineer knows that permanent gains in wealth and leisure are the by-products of rising efficiency, and cannot be created by government subsidy; that the way to cure unemployment is to create more jobs through research, thrift, and enterprise, by developing new products, by creating new industries, and by translating technical advances into reduced prices and wider markets. One quarter of all our employment today is said to be in industries which did not exist before 1880.

In our modern industrial society we thrive not merely through what we spend for goods quickly consumed, but through what we spend to put more men to work. On the average, it costs about $7,600 today to equip a worker for his job. The flow of new capital into investment has all but stopped, under a public policy of policing industry rather than encouraging it, and there is little prospect of healthy recovery until it is reestablished. The accumulated deficit of capital replacement and new investment of the last decade has been estimated at something like 150 billions. This is a vital matter to engineers. We spend the money which prepares jobs for men, directing it through construction and manufacturing channels into wages and purchases of materials, through which it ultimately is transformed into purchasing power for consumption goods. Engineers can thrive only when society thrives by multiplying its economic capacity.

A Critical Choice.—Facing tomorrow, you are facing the risks of decision which mark off the man from the boy. Heretofore, most of those risks have been taken for you by parent, school, or teacher. You will be tempted, in the spirit of the times, to prefer security to adventure. Most of you who do so choose will settle into mediocrity. If you have not had the privilege of going away from home to college, try for a job in some other town. Finish the job of growing up, if possible, out from under the wings of the family and the college which reared you.

The engineer does not shun risks, but he takes them prudently. It will be important to you to choose a field which challenges you, but one in which you can succeed by reasonable application and effort. Success is a habit, and not a lucky break. Education comes through success, in getting a taste of achievement which creates a craving for more. Men are lured to success, not driven. If you succeed it will be because you are spurred by inner drives rather than by outside rewards.

You will begin life in a competitive struggle, but with the odds in your favor. President Compton of the Massachusetts Institute of Technology, reporting on a study of 54,000 officers of 500 corporations, has stated that the college man is seven times more likely than a non-college man to become such an officer, but that an engineer is thirty times more likely than a non-engineer. The advantage is great, but it is well to judge the odds in the light of changing conditions. Going to college has become a generally accepted social habit, much as going to high school became twenty-five years ago. One young person in seven enters college today. Place beside this fact another, namely, that about one family in seven is above the income level of $2,400 per year, and it is plain that college-going has about caught up with preferred opportunity in our present society.

On Your Own.—The odds are with you, but they will not save you from the relentless sifting of the first five years out of college—the most critical period of an engineer's career. The shock of passing from college, where everything conspires for your development, to a realm of repetitive work has been likened to shifting a fast-moving car from high to low gear. It is a jolt to find that you are expected to go on with work after you feel that you have exhausted its learning possibilities, to find yourself and most of those about you using so little of what you learned in college, to have a boss who pushes you on output while you have to push him if you wish to learn anything. You will find that where a hundred men will learn under the organized routine of a school, or ten under the inspiration of a voluntary group, only one will keep driving ahead under his own power.

When you discover that you must now fight your own battle for self-development, you will be tempted to work for yourself rather than your employer. You will want to be a brilliant performer and to go places in a hurry, with dreams of a vice-presidency at thirty. Then some day you may awaken to the fact that too much zeal of this kind is holding you back, that you are really heading for one of those "lone worker" jobs which are blind ends in most organizations. At this stage, a good football player is likely to hold an advantage over a brilliant scholar. To get ahead on the main track, you will have to learn to put your organization first and adjust yourself to the tempo of group activity. You will have to get others to perform and not merely perform yourself. You will have to learn to judge men, fit them to their tasks, train them, iron out their troubles, check their performance, appraise and reward their work, and inspire them with organization spirit.

Surmounting Routine.—Educators of all sorts are observing that young engineers stand the shock of adjustment to the world of work better than college men in general. They have a clearer sense of direction. Fewer of them flounder. They know the worth of discipline. Sustained work is no novel experience, nor does physical effort appall them. Few are troubled with conflicts of personality. They accept economic realities. Employers have long since recognized these qualities, and the depression years have emphasized the engineer's preferred position. To you seniors facing tomorrow these are grounds for self-confidence, but not for over-confidence. The make-or-break test lies just ahead. The critical question is "Can you surmount routine?" Every good college of engineering trains men for careers of decision and action. Their opportunities are boundless, but they set their own limitations of achievement.

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NUMBER 5

Control Surveys for Flood Protection Projects in the Pittsburgh District

By THOMAS J. MITCHELL

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

FORMERLY ASSISTANT ENGINEER (GEODETIC), CORPS OF ENGINEERS, PITTSBURGH, PA.

IN the Pittsburgh District of the Corps of Engineers are a dozen or more widely scattered but closely interrelated flood protection projects. By means of the control network described in this article, surveys for all these projects have been developed on a single datum. The surveying methods adopted were dictated by the need for speed, accuracy, and economy, and included a number of novel features that should prove of value in similar work elsewhere. Of particular interest is the use made of the state coordinate systems.

ABOUT the middle of the eighteenth century a young engineer, George Washington by name, was returning to the Virginia settlements from an official call on the French forts near Lake Erie. When he arrived at the Allegheny River, at what is now Pittsburgh, he encountered the river in flood and had considerable trouble making the crossing. This may have been the introduction of the engineering profession to the problems of flood control on the upper tributaries of the Ohio River. Whether this surmise is correct or not, certainly floods have been an ever-present problem for Pittsburgh and vicinity since that time. In recent years much study has been given to these problems and excellent reports written, so that, when in 1936 it was possible to start work on remedial measures, the apparently feasible projects could be selected for immediate detailed study.

Of course, the first requirements were large-scale maps of the areas. The large number of projects, the necessity for speedy and accurate results, and time and financial limitations made these mapping surveys a job of some magnitude and complexity. The different projects were related to such an extent that it was apparent they should be developed on the same datum, and this made necessary the establishment of extensive horizontal and vertical control for the mapping program. All this work fell within the jurisdiction of the Pittsburgh District of the Corps of Engineers, U. S. Army.

The flood projects and projects related thereto, of the Pittsburgh District, extend from New York State on the north to well down in West Virginia on the south, and from the Allegheny Mountains on the east to Ohio on the west. No single plane coordinate system would be adequate to cover this area, and a multiplicity of plane systems on such closely related projects would be too troublesome to countenance. A geodetic system based upon the national control would perhaps have been the ideal solution, but from a practical standpoint neither sufficient personnel experienced in geodetic methods nor time for training the personnel in such methods was available, for after all geodesy is a specialized field of engineering, requiring training and experience comparable to any other specialized field. There are numerous geodetic approximations which are justifiably used in other parts of the country, but which did not promise to produce the results desired. What apparently was required was a system conducive to the use of plane coordinates by the majority of the personnel, and at the same time capable of geodetic refinements. Fortunately such a system was available.

A few years before, at the request of outside organizations and engineers, the U. S. Coast and Geodetic Survey had developed a system of plane coordinates for each state based upon standard mathematical projections. This was a very happy solution, for while its mathematical character enabled any desired refinements to be attained once the control points had been coordinated on the state system, auxiliary control and mapping could progress in the same manner as with ordinary plane coordinates. About this time the legislature and governor of the Commonwealth of Pennsylvania legalized the use of the state system, which gave it a unique legal position. The Pennsylvania and West Virginia systems are based on the Lambert conformal conic projection. That of New York State is based on the transverse Mercator projection. Precedents and methods for the use of these systems on engineering surveys preliminary to construction were developed by the personnel without difficulty as the need became apparent. There are two zones for Pennsylvania, and those, with the adjoining state systems, may necessitate a transfer between systems at certain localities, but such transfers would present no difficulties. Likewise the change from the local plane coordinates formerly employed in certain localities has been expeditiously made.

SECOND-ORDER ARC TIES PROJECTS TOGETHER

The working area, Fig. 1, was in general well adapted for the use of triangulation, supplemented by traversing, as the main horizontal control. A few of the projects had national arc stations in the immediate vicinity, and the control for these was turned directly off the national stations. Unfortunately, most of the projects were at a

CHANNEL IMPROVEMENT ON THE CONEMAUGH AT JOHNSTOWN, PA., IN DECEMBER 1938
An Unusual Type of Control Survey Was Used for This Project

considerable distance from control points, so a long arc was run spanning the distance between two national arcs and tied and adjusted to both. This new arc was read under second-order specifications with substantially first-order closures. Lines of this long arc (Fig. 2) form starting bases and check points for the triangulation schemes of several reservoir sites.

The individual figures of all the triangulation in the District are of the closed type; that is, there are two independent ways of computing length through each figure except for the rare occasion when it was not justified. It was difficult to foresee the possible future uses of the triangulation so it was made as rigid as economically feasible. The triangulation angles for the individual project were read as far as possible with Parkhurst direction theodolites, and the principal arcs entirely so. There were two direction instruments available and several repetition instruments. The direction instruments were used to the limit of their capacities for reasons of precision and economy.

The triangulation reconnaissance placed the stations to allow the observer to work from the ground or, when that was not possible, at the minimum height, to hold the cost of tower building as low as practicable, for on this job towers were unduly expensive. It was frequently found more economical to clear lines than to build above the obstructions. To determine the location requiring the least clearing, frequently flares were ignited and observed at night.

DIFFICULTIES IN OBSERVATION

Observing conditions differed widely over the area. In the sector to the east of Pittsburgh the mountain front seemed to trap the smoke from the metropolitan manufacturing district and produced an atmospheric condition that made daylight observing impractical except on very short lines. Hence nearly all this observing was done at night. Nearer the Great Lakes, daylight visibility was more nearly normal. The observing conformed to specifications for second and third order work of the U. S. Coast and Geodetic Survey and the American Society of Civil Engineers, Manual No. 10, "Technical Procedure for City Surveys." Most of the equipment was designed by our organization and manufactured in our shop.

Night observing was not without its complications. Light-keepers, under the set-up, had to be obtained from WPA rolls, and as they were required to be residents of the locality in which they were working, they were not generally subject to transfer between jobs. This necessitated the periodic training of new men. Normally, the observer communicates his orders with lights by means of the International Morse Code. For the Pittsburgh District work, transceivers (portable short-wave radio sets, capable of transmitting and receiving) were designed experimentally to supplement the lights as the medium of communication. The design is of limited application, for it was necessary to restrict the sending range so as to avoid interference with the numerous amateur stations in the vicinity. It is planned to use transceivers of a similar design also for flood emergency communication.

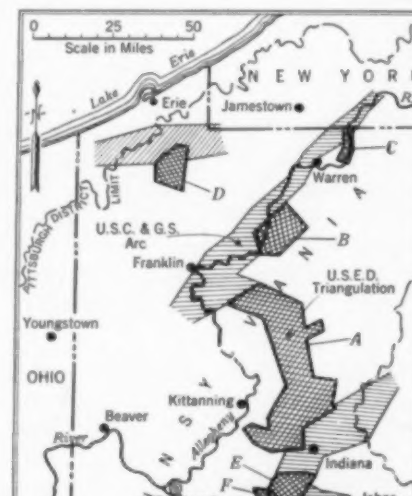


FIG. 1. AREAS COVERED BY TRIANGULATION
A, U.S.E.D. Main Arc, Including Redbank Creek, Mahoning Creek, and Crooked Creek Projects; *B*, Tionesta Creek; *C*, Upper Allegheny River; *D*, French Creek; *E*, Conemaugh River; *F*, Loyalhanna Creek; *G*, Stony Creek; *H*, Youghiogheny River

Each individual arc presented a different problem in adjustment although all were rigidly adjusted finally by a method of least squares. The speed with which the jobs were commenced and conducted did not permit the ideal method of obtaining and adjusting the control before detail operations started. All phases of the work had to be carried on at the same time, and it was not until well along in the program that the progress of the triangulation placed it ahead of the plane tables. Usually at first a base line was measured, an azimuth observed, and triangulation started. At closer than ordinary intervals other bases were measured. The first surveys for topography on each project were made on an arbitrary local plane datum, the starting point of which was designated N = 100,000, E = 100,000. As soon as the angles of the figures between base lines were read, they were in general adjusted temporarily by an approximation which closed the triangles and made the sides consistent. Where the figures approach closely to a square, the corrections approximate those obtained by least squares, but for figures of other shapes the corrections are unnecessarily large. While the adjustment is rigid, it does not compare favorably over an extended

arc with the least square adjustment. It does have the advantage of giving consistent control points quickly and economically for temporary use. In some cases, when the local triangulation was finished before the connection to the national arc was ready, the local arc was adjusted as a unit by the method of least squares and later adjusted to the national arc by means of length, latitude, longitude, and azimuth equations.

As soon as the figures forming the connection to the national arc were completed, each system was adjusted as a whole by the method of least squares. In this way there was a good comparison between the different methods of adjusting. The rigid least square method gave uniformly better probable errors and positions than other adjustments and proved to be more economical than had been anticipated. Adjusting a system as a whole presents a formidable lineup of equations, but their solution with the aid of a modern computing machine is not particularly difficult.

As soon as the arcs were adjusted, geodetic positions were computed and then these positions were transferred to the state coordinate system. To get a comparison of costs, triangulation for two of the projects was adjusted and computed directly on the state coordinate datum. The results seemed to indicate that except for short lines in arcs predominately east and west, on parts of the projection where the scale correction was negligible, there was no particular advantage in this method.

Each project formed a convenient filing unit. A drawing was prepared showing the triangulation scheme to scale and the names and descriptions of stations and bench marks. Thus a single blueprint would contain all the field information. The office results were so tabulated that they too could be quickly blueprinted.

In the early part of the control program, triangulation base and control traverse measurements were made with standardized steel tapes. Very good results were obtained, as cloudy weather makes this part of the country peculiarly suited to the use of steel tapes. Ordinary base-line precautions as to temperature, slope, tension, and end-marking were observed. Later, invar tapes and equipment were secured. On one base line of aver-

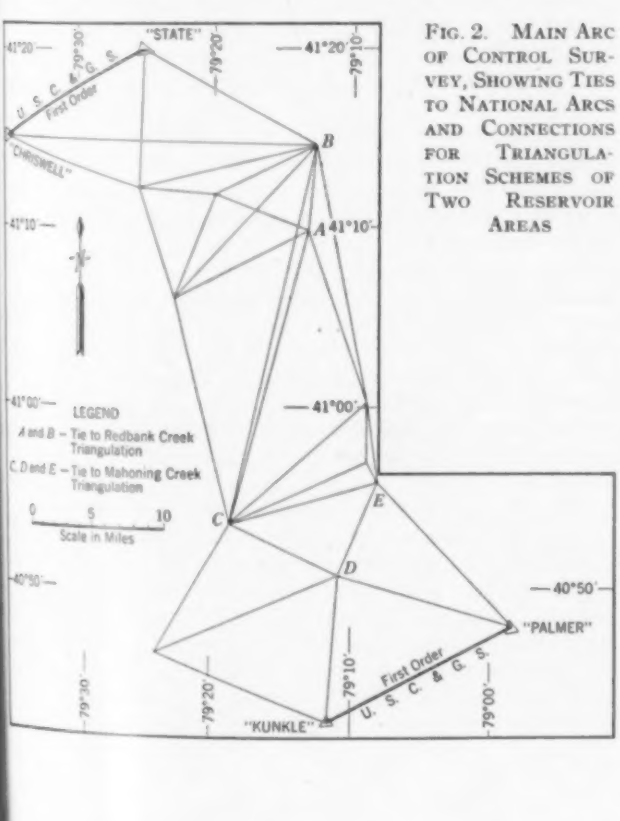


Fig. 2. Main Arc of Control Survey, Showing Ties to National Arcs and Connections for Triangulation Schemes of Two Reservoir Areas

Upstream View of the Tionesta Creek Dam Site

age type a comparison was made between the cost of obtaining the precision needed with steel tapes and with invars. The invars proved to be 40 per cent cheaper.

An interesting and unusual project developed at Johnstown, Pa. Johnstown has had in the past an unenviable reputation for disastrous floods. The project designed to remedy the situation is a channel improvement of the Conemaugh River, Little Conemaugh River, and Stony Creek. On nearly all these river stretches, horizontal clearances are very small, especially in the industrial and business areas. On account of this limited leeway it was decided to make the layout control of a precise nature. The terrain was not suited to arc triangulation so a precise traverse was run on both banks of each creek as near to the construction limits as possible. Cross ties were made at intervals of not more than one-half mile by measuring across bridges or by triangulation.

The linear measurements consisted at first of two runs in opposite directions with an invar tape, and later of one run when results showed that the second run was unnecessary for the desired precision, which averaged 1 part in 100,000 over each of the three rivers. Intermediate closures were larger but averaged about 1 in. per mile of perimeter. These two traverses, separated by from 200 to 400 ft, gave a good opportunity to study the rigidity of precise traverses. The traverse angles were determined for the most part by a repetition instrument graduated to 10 seconds. The invar tape was supported on light-weight chaining tripods. No attempt was made to secure the ultimate in high precision, and practically no measurements were repeated. The city of Johnstown has a control system based upon three triangulation stations of the national arc, and the primary traverses were checked against this control with satisfactory closures.

This method of traversing is relatively inexpensive for the results obtained. The necessity for such precision in construction surveys might be questioned, but in this job, where inches in clearance were important, the normal fluctuations in even an excellent steel-tape traverse would have been troublesome and an attempt to raise the precision of a steel-tape traverse to the desired level would have been unduly expensive.

Where it was economically feasible, the control arcs for individual projects were connected to more precise arcs by line ties at both ends. Where this procedure was not feasible, position ties or base-line and azimuth ties were used.

For azimuth observations a direction theodolite with a sensitive striding level was used. The cost of such a precise determination, so far as field work is concerned, is substantially the same as that of an observation with an

ordinary transit. The computing costs are higher, but not excessively so. A second-order precision with a probable error of less than two seconds is easily obtained, while with an ordinary transit the coarseness of graduations and doubt of the verticality of the vertical axis throws suspicion on results that apparently agree closely. Triangulation checks seem to bear out this suspicion.

When traverses have to be started before the triangulation control is finished, a precise azimuth observation enables the results to be transferred later to the state coordinate system with a minimum of cost and effort, for the mapping angle can usually be obtained by scaling the longitude of the starting point from the quadrangle sheet, and the geodetic azimuth changed to the grid azimuth. The resulting corrections to the local coordinates are then constants. With an azimuth of inferior precision there is an azimuth swing to be corrected, which leaves the expensive alternatives of recomputing the traverses or of preparing and using correction tables.

CONTROL FOR INDIVIDUAL RESERVOIR SITES

The control for the reservoir sites, as illustrated in Fig. 3, consisted first of a triangulation net whose stations were spaced according to the terrain; that is, in timbered areas where the building of high towers would have been necessary, the stations were widely spaced and the traverses connecting them were correspondingly tightened. In cleared country the stations were spaced correspondingly closer together. This close spacing was of material help in determining points for control of aerial photographs, which were used extensively.

Next a traverse for controlling the plane-table work was run along the bottom land of the rivers. Where the valleys were comparatively wide, frequently more than one control traverse was run. The traverses were connected to and adjusted to the triangulation at frequent intervals, so that not only the closing ratio, but also the total closing error would be held to the economic minimum. Additional traverses for plane-table topography were tied to the first traverses. Finally, traverses were run near the flow-line contour and adjusted as the others. The last traverses were especially useful in controlling the property acquisition surveys on the accepted projects.

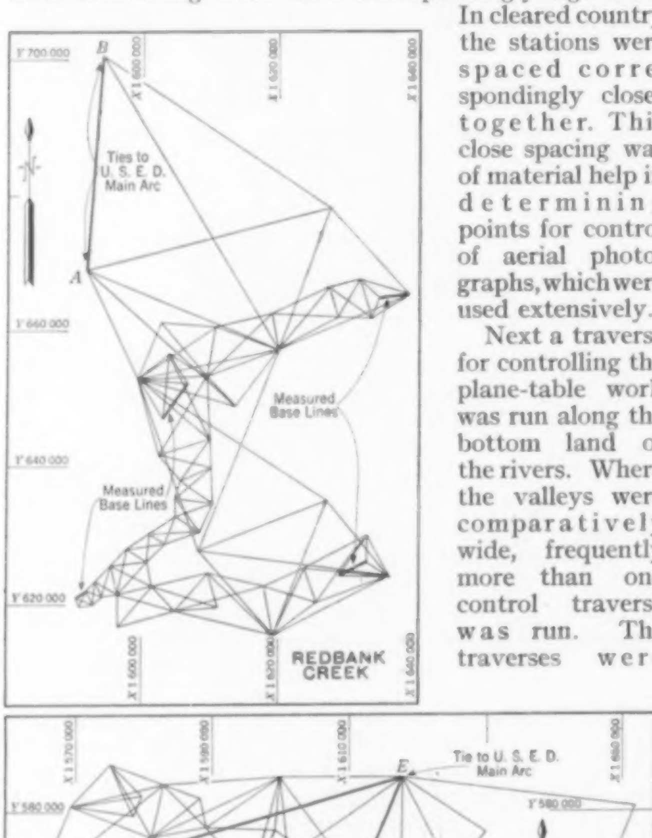


FIG. 3. LOCAL TRIANGULATION NETS FOR CONTROL OF TWO RESERVOIR AREAS

See Fig. 2 for Relation to Main Arc

In the interest of uniformity, specifications were written governing the different types of traverses. Incidentally, loop traverses were studiously avoided due to the fact that a suitable closure leaves entirely unknown the magnitude of actual position errors in the loop. These uncertainties are caused by so-called constant errors operating during the linear measurements. The specifications conformed in large measure with the unpublished specifications of the Committee on Control Surveys, Surveying and Mapping Division, American Society of Civil Engineers, with which some of the members of our mapping organization had had committee experience.

AN 80-FT STEEL FIRE TOWER USED AS INSTRUMENT SUPPORT

Note Observing Tent at Top

All traverses were computed on the state coordinate system and coordinates were listed for all the stations. Where a traverse was at an elevation above the contemplated high water line, it was monumented at suitable intervals to give readily both position and azimuth. These marks will furnish additional control for future surveys.

The areas were relatively well provided with precise bench marks of the national scheme. Reliable railroad and highway bench marks were also available. Intermediate bench marks for topography were run with Wye or Dumpy levels.

Many special assignments developed during the course of the work. One was of especial interest on account of its precision and possible application to work of a similar nature. It consisted of checking movements on and about the Tygart Dam, one of the largest concrete gravity dams in the eastern part of the country. The specifications called for detecting movements of the magnitude of $1/8$ in. on the structure and on the surrounding hillsides. This job necessitated new methods and a redesign of the equipment used, in order to obtain the desired precision. A description of this work is beyond the scope of the present article.

The control work of the Pittsburgh District was carried on by the Surveying and Mapping Section, headed by Payson A. Perrin, chief of the section, under the immediate direction of Floyd W. Hough and the writer. Lt. Col. W. E. R. Covell, Corps of Engineers, U. S. A., is District Engineer. All are members of the Society.

Time Studies on Heavy Construction

Third of a Series on Modern Construction Tools and Practice

By Frank C. Wardwell

Associate Member American Society of Civil Engineers
Senior Civil Engineer, TVA, Chickamauga Dam, Chattanooga, Tenn.

When time studies of heavy construction operations were first undertaken by the Tennessee Valley Authority some four years ago, there was considerable doubt as to their value in cutting costs or speeding up production. Today they have proved their worth many times over, and the time-study crew has become a key unit in the field organization on several projects. As a result the cost engineer, often merely a "collector of dead records," is effectively exercising on TVA projects what is properly his chief function—assistance in reducing costs. Time studies such as those Mr. Wardwell describes are applicable to an almost limitless variety of operations, and his article may be read with profit by engineers and contractors on all types of construction. The present text is an abridgment of his paper on the program of the Construction Division at the Society's 1939 Spring Meeting.

We are occasionally told of old contractors who, when bidding on proposed work, simply walked over the site, calculated their bids on the back of an old envelope and, when awarded a contract, computed their profit—or loss—by checking bank balances at the end of the year.

Most of the contractors who worked in this free and easy way are here no longer. Some have died of heart failure; some have retired by way of the bankruptcy courts; perhaps a few may be wintering at Palm Beach. Contracting—and all major construction—is being placed more and more on a firm business basis. And for this reason business principles in construction are vital as never before. Among such essential principles is a thorough knowledge of operating costs.

To obtain all the necessary facts on any large business, a complete accounting and cost organization is essential. The accountant must obtain such data as to insure that all expenditures are for the purpose intended, and that bills have been paid only after proper safeguards have been observed as to inspection, delivery, and so forth. The cost engineer must correlate the expenditures with the progress of the work. He must be able to analyze all work done and to accurately determine the cost of each of its component parts. He must know how this cost compares with the original estimate and contract price, and with the cost of similar work done elsewhere.

The duties of the accountant and cost engineer may seem to overlap to a considerable extent. Nevertheless, they are separate fields. In general it may be stated that the accountant certifies that the expenditures have been made correctly in accordance with established procedure, while the cost engineer determines the manner in which the money was spent, its relationship to the quantity of work performed, and the reasonableness of the expenditure.

Many of the duties coming under the supervision of the construction cost engineer are routine. He must distribute the job payroll in accordance with work performed, code all invoices to the proper accounts, and see that all supplies are charged out to the proper place. He must distribute the large variety of overheads and the operating cost and depreciation of equipment; and see that such distributions are properly entered in the accounting ledgers in direct relation to the work performed. He must prepare, periodically as needed, analyses of these ledgers and compute unit costs, to keep the management informed of the financial condition of the job. He must see that proper records are kept of the methods used in operations, so that sufficient information is available to explain all unusual costs when final records are prepared.

But important as such routine duties may be, they in no way cover the real function of cost work. They are in fact only working tools used in carrying out more important assignments which, if rightly performed—and utilized—will repay, with dividends, the entire expense of the department.

The cost engineer's chief function on the job should be to assist in reducing the cost of the work. This can only be done by careful study of construction operations, by making prompt and thorough analysis of job conditions, and by calling attention to those items that are out of line with previous costs or that show possibilities of still further reduction. He should analyze reasons for delays, the cost of such delays, and suggest methods of improvement; he should prepare comparative estimates of various proposed methods of performing work and should have the opportunity to present his analyses to the job management before the method to be used is decided upon. His ability to analyze all the factors entering into costs, and his familiarity with construction details, make his judgment in cost matters equal to or greater than that of any other man on the job.

WHY COST WORK SOMETIMES FAILS TO PRODUCE RESULTS

Obviously, if such duties are expected of the cost engineer, he must be a man of mature judgment and much construction experience—and he must be provided with an adequate staff. Job management is often to blame for the failure to profit by cost work. Frequently, the salary scale allotted to the job of cost engineer is entirely too low to secure a man capable of performing more than the routine duties of his position. More often, a well-qualified man is selected but given only sufficient force to perform the necessary routine duties, with the result that he has no time for his more important functions. Again, it sometimes happens that the cost engineer is able and ready to supply the data needed but is not called upon to supply it, or is rebuffed when he offers it. All too frequently, any suggestions for the betterment of costs are looked upon simply as criticisms of those responsible for existing conditions and treated accordingly. Where this is the case, the cost engineer is of little worth to his company beyond that of a collector of dead records.

Among the methods that a cost engineer can use in determining and controlling construction costs, and one which has been almost entirely overlooked, is the use of time studies. Frederick Taylor's *Time and Motion Study*, published nearly fifty years ago, called to the

attention of building contractors the economies that were possible by careful analysis of various factors entering into the labor cost of construction. His ideas received much attention in various industrial plants, but much less attention in the construction field.

About 1900, Frank B. Gilbreth, an engineer and contractor, made extensive studies along the same lines in the building industry, particularly in bricklaying. In this work he was ably assisted by his wife Lillian, a brilliant economist. Together they worked out the basic principles of motion study in so far as it applied to the building industry, and Gilbreth used time studies in his own contracting business with considerable profit. Much was accomplished by these pioneers, Taylor and Gilbreth, and the papers written by them are interesting additions to the economic side of engineering.

Some difficulties have arisen over the use of time studies in industry. Union labor has seldom been favorable to them, and undoubtedly they have also suffered from the blight of the so-called "efficiency expert," who has recommended practices that have not always been feasible. These reasons account, perhaps, for the fact that little use has been made of time studies on heavy construction. Also, it must be realized that construction activities do not lend themselves in quite the same way to time study as do certain factory operations which can be standardized, and which extend with little change over long periods of time. For this reason, the use that has been made of time studies in the Tennessee Valley Authority during the past four years should be of interest.

TIME STUDIES ON TVA PROJECTS

The first of these investigations was a complete study of drill-bit performance, begun at Norris Dam in November 1934. (Details of this investigation are given later in this article.) So valuable were the results that shortly thereafter a regular time-study group, consisting of three engineers, was organized to make similar investigations of other construction operations. From May to September 1935, this group made studies at Norris Dam of the erection of panel forms, power-house structural steel, cement handling operations, and the operation of placing the closure gates. It was then transferred to Wheeler Dam, to make studies of panel form erection, line drilling, welding operations incident to intake gate erection, and some other minor operations. In March 1936, the party was moved to Pickwick Landing Dam, continuing studies of panel form erection, terminal unloading facilities, the cement unloading plant, and concrete placement with the large gantry cranes. Four months later it was transferred to Chickamauga Dam, where it is still engaged.

During this last period of approximately 33 months, more than 30 extensive studies and many shorter ones have been made. Among these the following representative subjects serve to illustrate the operations that have benefited: (1) erection of panel forms for lock, spillway, and power house; (2) operation of carrying scrapers of various capacities and arrangements; (3) crushing plant operation; (4) conveyor and mixing plant operation; (5) miscellaneous hauling equipment; (6) placing of rolled earth fill; (7) rock excavation; and (8) miscellaneous rock drilling operations.

The variety of operations listed in the foregoing history of TVA time study work indicates the extent to which it seems desirable to carry such investigations. In general, time studies can be used profitably in the following classes of operation:

1. Operations of a continuous or frequently recurring nature
2. Comparisons of equipment performance
3. Selection of the proper operating group
4. Quick determination of unit costs for estimating or comparing construction methods

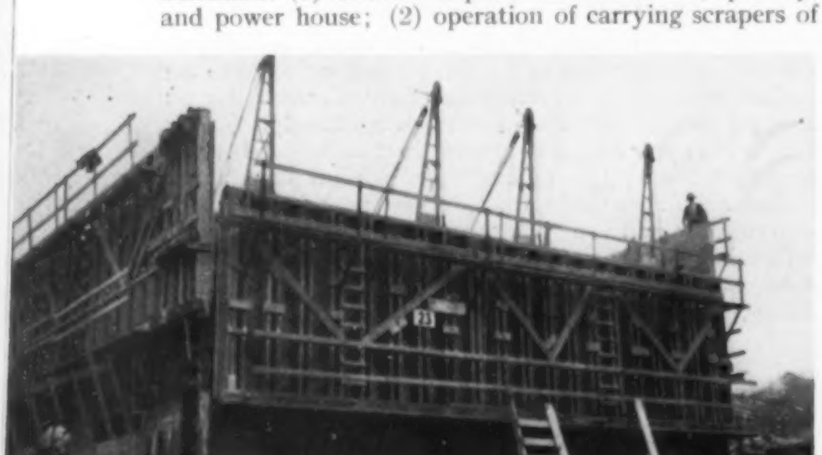
Among the studies in the first group, that of form erection has been the most extensive. On a project similar to Chickamauga, nearly two million square feet of forms are required, and the cost will be several hundred thousand dollars. This form erection extends over a considerable period of time, and any savings in unit cost of erection and stripping that can be pointed out at the beginning of construction will be multiplied many times over before erection is complete. Other items in the same classification are earth and rock excavation and the placing of rolled earth fill.

Studies that fall into the second group are those of drill-bit performance, wagon-drill production, and carrying scraper performance. Such studies have given very valuable information for use in the writing of specifications and the purchase of the right type of equipment for some particular operation. Practically all contractors have a very definite idea of the type of equipment that gives best performance under certain conditions, but it sometimes happens that these ideas are founded on very little definite information, and that more exact study will show them to have little justification.

Studies in the third classification have to do with the selection of crews for mixing plants, crushing plants, and so forth. For a "standard" crew, certain definite personnel is of course essential. In addition to this fixed personnel, it is customary to provide a small force for routine maintenance work, repairs, and emergency operation. This latter force frequently gets out of line with actual needs and remains unnoticed for a considerable period. Substantial savings have been made in such an operating crew by a careful study of personnel from time to time. It is often found that such forces are only infrequently needed and that the regular repair force can handle these operations much more efficiently and just as promptly. On the other hand, it sometimes happens that plant delays involving a considerable number of men can be much reduced or entirely avoided by the employment of a single man in some key position, and this extra man will actually produce economies in operation. Only by very careful analysis of the operations involved can such economies be determined.

As for the fourth class of studies, the time-study crew can frequently be used to obtain accurate approximations of unit cost, needed for some particular purpose, in much less time than the regular cost department in the usual routine of its operations. Careful observation of the operation of a single shift and a comparison of the production on this shift with production on other shifts will provide unit costs sufficiently accurate for most purposes.

The success of time studies on any job will depend in large measure on the men selected to conduct them. Such men should have engineering training, and expe-



RAISING PANEL FORMS AT CHICKAMAUGA LOCK BY MEANS OF PORTABLE ALUMINUM A-FRAMES

Such Operations Were the Subject of Intensive Time Studies

rience on engineering and construction work. A keen sense of observation is necessary, and they should have tact and speak the language of construction men. The time-study crew, as organized for TVA work, was headed by an engineer of mature judgment, thoroughly familiar with construction methods, and able to win the confidence of foremen and laborers and to keep their good will while at the same time pointing out ways in which delays could be reduced and construction progress expedited. Under this man were two or three younger engineers, each of whom had had some experience in construction work both in field and office. They were the type of men who could establish themselves on a firm footing with the men on the job and secure their cooperation and assistance, and one of the men was chosen because he had training in the preparation of engineering reports.

A few specific examples will serve both as an example of the manner in which the time studies have been conducted, and of the results that have been accomplished.

SELECTING THE MOST ECONOMICAL SIZE OF SCRAPER

The Authority owned some twenty carrying scrapers of 12-cu yd capacity. When additional equipment became necessary for placing rolled earth fill at Chickamauga, consideration was given to the purchase of units of greater capacity. However, the manufacturer was unable to give conclusive evidence as to the probable economies of operation of the two different sizes of units operating under the conditions existing at that site.

Accordingly, an 18-cu yd unit was obtained on a rental basis and placed in operation along with a similar machine of 12-cu yd capacity acting as a single unit, and also with two 12-cu yd machines operated in tandem with a single power unit. Stop-watch records were taken of the time required for each machine to load and to dump, and of the travel time both loaded and empty under various lengths of haul, different weather conditions, and with different operators. Operating costs and repairs were carefully compared on a unit cost basis; depreciation of equipment over its expected life was given full consideration; and every factor that influenced costs in any way was thoroughly studied. The study extended over a period of 16 days and covered production during 48 seven-hour shifts, during which 56,393 cu yd of compacted fill were placed. The average hourly production per machine during the study was 61.6 cu yd per gross hour, with an average haul of 1,140 ft.

The study indicated that the most economical of the three machine units gave production costs 12.2 per cent lower than the next lowest, and 18.6 per cent lower than the third. On the basis of about 2,000,000 cu yd of fill remaining after the completion of the study, a saving of approximately $14,000 between the most economical and the next most economical unit was gained. It should be pointed out that had a decision been made using operating costs alone, without considering all cost factors, an entirely different and wrong conclusion would have been reached.

STUDIES OF WAGON DRILLS AND DRILL BITS

Late in June 1936, the Authority received identical bids from six manufacturers on wagon drills for use at Chickamauga. Rather than select by lot the type of equipment to be purchased, the Authority decided to make exhaustive tests to determine the most economical drill when operated under actual field conditions at Chickamauga.

Twelve wagon drills were obtained from the six manufacturers and given a complete field test. Careful records were kept of the hourly rate of drilling under varying field conditions. Drillers were rotated to different drills to eliminate the human factor. Materials and supplies were computed, and the costs of repair parts and their installations were added. Careful measurements were made of air consumption, both during drilling and in cleaning holes, and the cost of this air was computed. Records were made of the replacement of rotating parts, both in drilling and pulling steel. Every possible factor that influenced drilling cost was considered.

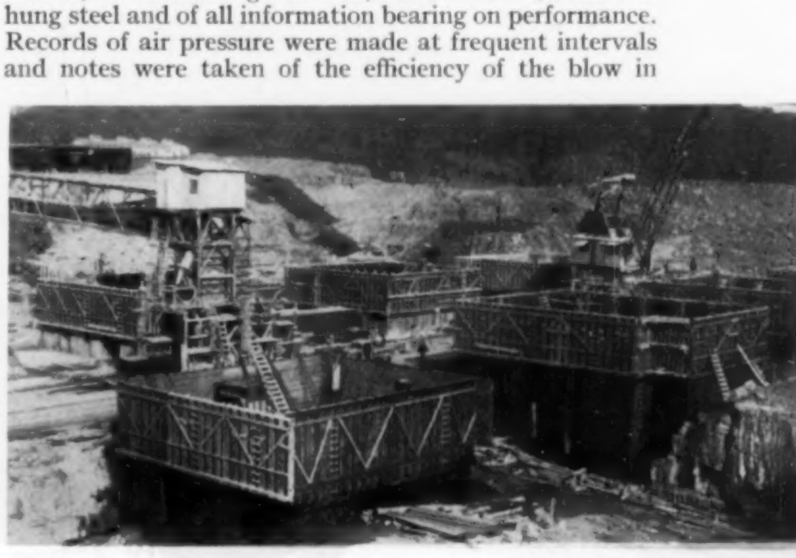
The study was made with the cooperation of the sales representatives of the six manufacturers, and each manufacturer was furnished with full information on the performance of his drill as the tests progressed, but was given no data regarding his competitors. After the test was started the manufacturer was not permitted to make changes in his equipment, or the manner in which it was operated. So far as we know, every manufacturer was satisfied with the fairness of the test.

These tests continued for 97 days, during which time a total of 146,748 lin ft of drilling was completed. The results demonstrated that two drills were of almost identical efficiency, and that they gave a unit drilling cost about 6.8 per cent below that of the third drill and nearly 34.8 per cent below that of the drill giving the poorest performance. Consequently one of these two drills was adopted as standard equipment at Chickamauga and the other as standard equipment at Hiwassee. The savings as represented by the difference in drilling cost between the two best drills and the next best was approximately $0.0044 per lin ft, or $4,000 on the 900,000 lin ft of wagon drilling at Chickamauga. Here again the conclusions reached after full study were quite possibly very different from those that might have been drawn from "obvious" but incomplete data.

The manufacturers evidently profited by the test, for one of them obtained from the chief of the time-study party 28 specific recommendations for the improvement of his equipment, and we are told that 21 of these have now been adopted.

Mention has already been made of the drill bit studies made at Norris Dam in 1934–1935. At that time the use of detachable bits on heavy construction operations was not common practice. The conventional forged bits were in general use and it was not usually admitted on construction jobs that there was any proved economy in the use of detachable bits. Manufacturers' claims were conflicting and little seemed to be known of the relative merits of various types.

Extending over a period of six months, these tests covered observations on forged bits, and on detachable bits from three leading manufacturers. Tests were made under a variety of representative field drilling conditions. Four different makes of wagon drills were used during the time the investigation was in progress. Performance records were kept of new bits and of first, second, and third regrind bits; also of all broken and hung steel and of all information bearing on performance. Records of air pressure were made at frequent intervals and notes were taken of the efficiency of the blow in



FORMS, CONCRETE CONVEYOR, AND HOPPER AT CHICKAMAUGA LOCK
Concrete Placed by Gantry Cranes

PLACING ROLLED FILL AT CHICKAMAUGA, WITH SCRAPERS OF VARIOUS CAPACITIES OPERATED SINGLY AND IN TANDEM

Selection of the Most Economical Unit Was Made Possible by a 16-Day Time Study

handling cuttings. The holes, which averaged approximately 30 ft in depth, were in general drilled with $1^1/_4$-in. hollow round steel.

This study quite definitely established the superiority of the detachable bit over the conventional forged bit when used under the conditions existing at Norris and similar TVA jobs. This was true both under average conditions and also where favorable conditions as to transportation and nipping costs tended to favor the forged bit. A difference in cost between the most economical and the least economical detachable bit was also noted.

During the entire period, representatives of the companies manufacturing the bits were present and assisted in every way possible in the experiments. They were able to make many valuable suggestions toward improving drilling costs, and also to collect data leading to the improvement of their product.

BOOSTING THE OUTPUT OF THE MIXING PLANT

Shortly after operations were started at the Chickamauga mixing plant, a very careful study was made of the plant performance and duties of each member of the operating personnel. This study was undertaken to determine whether theoretical maximum hourly production could be raised by changes in plant design and also what steps could be taken to keep actual production more nearly up to the theoretical maximum.

A total of 535 stop-watch readings were taken of the time required by the operator in batching the mix, and about the same number of observations were made of the time required for discharging the 2-cu yd mix, the lapse of time between batches, the charging time, and the time required for mixers to dump their charge and return to a horizontal position, and of other similar operations. From these data the average time for each operation was obtained, and it thus became possible to determine the effect on the mixing cycle of any change made in any of the operations. It was found that only minor changes of plant were required to bring about a reduction of 28.9 seconds in the length of the mixing cycle, without in any way reducing the mixing time. The effect of this was to increase the maximum hourly production of the plant from 88.52 to 100.38 cu yd—an increase of 13.4 per cent.

A personnel diagram was also prepared showing the respective locations of each employee in the plant on each shift, his badge number, rate of pay, and exact duties. Presented in this form, differences in the personnel on the three shifts, their duties, and rates of pay became at once apparent. These differences, together with a tabulation of average shift production over a period of time, enabled the superintendent to make a rearrangement of employees and their duties which had a decided effect in cutting unit production cost.

Similar studies were made of the quarry and crushing plant, and of the plant used in placing concrete, with similar results.

CUTTING COSTS OF FORM ERECTION AND STRIPPING

Probably the most extensive use of time studies on Tennessee Valley Authority work has been on form erection and stripping, particularly on panel forms. Form costs represent from 30 to 40 per cent of total concrete costs on low dams and locks, and from 15 to 25 per cent on high dams. On the six TVA dams already constructed or well along toward completion, the forms total over 12,000,000 sq ft of contact area. Time studies of form erection and stripping have been made on five of these six dams.

The surveys determined the actual average time required for each of the many operations entering into the setting of panel forms, removing these forms, and raising the panels to a new elevation. The cause and extent of all delays to form work were surveyed. Every factor which in any way affected form cost was carefully studied. Even the type of workman employed on form building was considered. On form removal the economies of form stripping by carpenters and carpenters' helpers was determined.

Among the many suggestions toward more economical form costs, the following are representative examples:

1. Changes in the methods of stocking and handling form material to reduce non-productive time of form crews.
2. Redesign of form hardware to reduce breakage of bottom tie lag bolts.
3. Change in design of panels so as to retain all reusable hardware in position while forms were being stripped and raised to a new position.
4. Redesign of channel lugs and assembly, thereby eliminating welding of lugs where extra bracing was required.
5. Changes in removal methods to reduce panel form repair cost to a minimum.
6. Elimination of split crew to increase efficiency of erection.
7. Purchase of sufficient form stripping tools to eliminate time lost in waiting on tools.
8. More careful oiling of form hardware to eliminate delay in the removal of "frozen" bolts.
9. More careful setting of form anchors, by means of a template, to speed up the setting of form braces and reduce the various lengths of bracing required.
10. Use of safety scaffolds to speed up erection.

This list could be extended almost indefinitely, but it gives a general idea of what was accomplished. Through these changes in design and methods of operation, form costs were unquestionably reduced.

One incidental effect of all these time studies, not heretofore mentioned but having a very important relation to cost, is the marked increase in the individual production of the men themselves while under observation. On those operations where records of production per shift were kept both before time-study observations were started and during the study, almost without exception production immediately increased on the first day the study was in progress. The men realized this increased production and often remarked on it to the time-study crew. Frequently it resulted in considerable good-natured rivalry between different crews or between different shifts. So far as we have been able to determine, this increased production by the workmen never resulted in any ill feeling among them toward either the time-study crew or the management.

The Southwest Sewage Treatment Works

Chicago's New Activated Sludge Plant Is Largest in the World

By William H. Trinkaus

Member American Society of Civil Engineers
Chief Engineer, The Sanitary District of Chicago, Chicago, Ill.

Simplicity, compactness, and economy from the standpoint of both construction and operation characterize the design of the Southwest Sewage Treatment Works of the Sanitary District of Chicago. Among the novel features are a mechanical method for drying and incinerating sludge, the use of air lifts in place of pumps for providing the necessary head for sludge return, and the plan adopted for smoothing out the peak demands on the sludge-handling equipment. A complete description of this new 400-mgd plant is given by Mr. Trinkaus in the accompanying article.

Early in 1935, construction was begun on the Southwest Sewage Treatment Works, the last of the four major projects in the program of sewage treatment of the Sanitary District of Chicago. This is the largest activated sludge plant in the world and embodies many novel features specially developed by the engineering staff of the Sanitary District. When the Southwest Works is completed and placed in operation, about July 1, 1939, sewage from approximately 99 per cent of the human population (estimated at 4,684,000) and substantially all the industrial wastes (population equivalent estimated at approximately 1,810,000) originating within the Sanitary District will receive some form of treatment—complete by the activated sludge process at the North Side, Calumet, and Southwest works; and partial by settling in Imhoff tanks at the West Side Works.

In the original project studies, a separate site was indicated as being necessary for the Southwest Works, owing to the large area required for digesting the sludge and drying it on under-drained sand beds. However, as a result of exhaustive experiments made by the Sanitary District, a mechanical method was originated and perfected for sludge handling by dewatering, drying, and incineration, which required a very modest area. Thus it became possible to locate the plant on an unused portion of the West Side site directly west of the existing West Side Works. The estimated tributatory human population to be handled at the Southwest Works as of 1940 is 1,277,000, together with industrial wastes equivalent to a human population of approximately 1,185,000, largely originating in the stockyards and packing-house district. The initial installation is designed for an average sewage flow of 400 mgd, with a maximum 50 per cent in excess of this. Long-time experiments at the North Side Works indicate, however, that the new plant will probably run satisfactorily at a continuous rate considerably in excess of the design average rate. A cross connection between the West Side and Southwest intercepting sewer systems at the plant site makes possible the diversion of West Side sewage in any desired amount to the Southwest Works. Thus the Southwest Works can operate continuously at its maximum load and give complete treatment to the largest possible volume of sewage, and the remaining West Side flow can be handled at the existing West Side Works, where at present only partial treatment is provided.

The Southwest Works (Fig. 1) consists of a combined sewage pumping station and blower-house, preliminary settling tanks, two batteries of aeration and final settling tanks, sludge concentration tanks, and a sludge disposal building adjacent to the pump and blower-house. Studies of future population growth indicate that eventually an average capacity of 1,200 mgd may be required to provide for the ultimate development of the Southwest Side area and to cover complete treatment for settled sewage from the West Side Imhoff plant. The plant units are therefore designed for extension to provide this capacity.

The plant site is at an average ground elevation about 10 ft above the level of the main channel of the Chicago

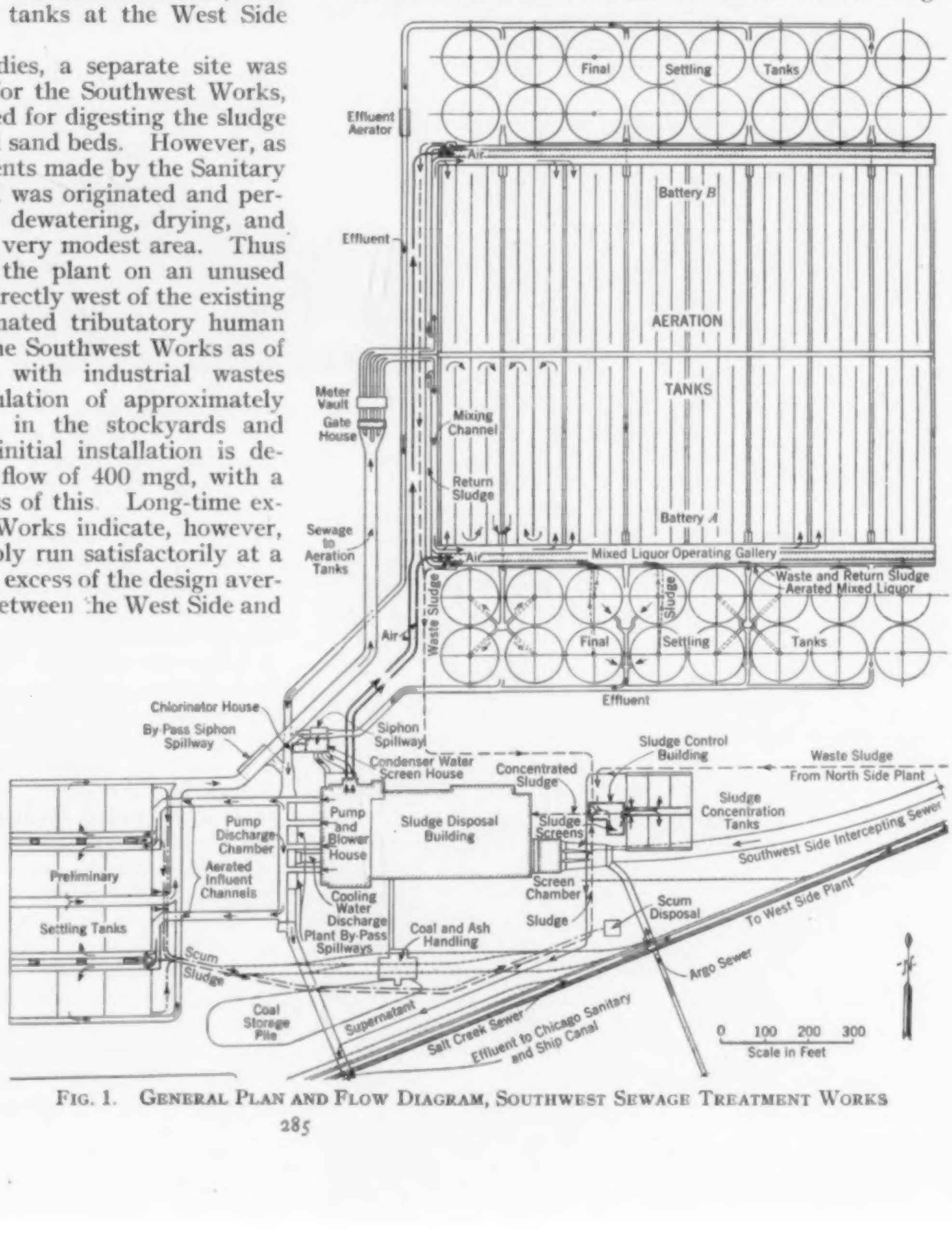
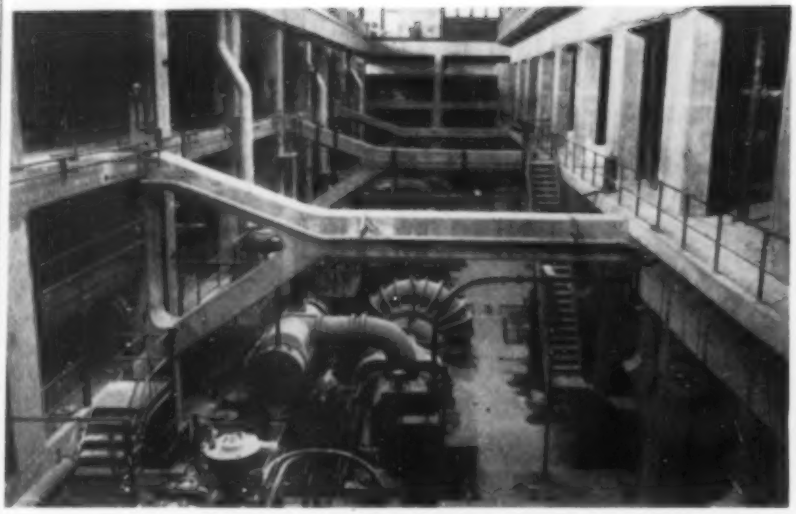


Fig. 1. General Plan and Flow Diagram, Southwest Sewage Treatment Works



THE CENTER BAY OF THE PUMP AND BLOWER HOUSE
Blowers and Turbo-Generators Are on Upper Floor at Left

Sanitary and Ship Canal, into which the plant effluent discharges. The total fall through the plant, from pump discharge to plant outfall, is also about 10 ft. From a practical standpoint, the entire plant could have been built wholly below the existing ground level. Comparison of the cost of power for pumping to different levels with the estimated fixed charges on the cost of excavation and pile foundations at corresponding elevations showed, however, that it was cheaper, in first and annual costs, to build the plant at an elevation about 12 ft higher than that determined by hydraulic considerations alone. In so doing, a large volume of deep excavation was eliminated. Cuts and fills were approximately balanced. The problem of adequate surface drainage was also greatly simplified, since final grades around the various plant structures are at or above existing ground levels. Subsurface conditions required no special treatment, except at the northwest corner of the aeration and final tanks, where the presence of running sand and peat necessitated the use of piles or fill concrete, depending on the depth to sound bearing material. The bottoms of the preliminary settling tanks were so close to the surface that pile foundations were used over the entire area.

Two 5,000-kva, 6,600-v generators supply energy for auxiliary motor-driven equipment and for plant lighting. These generators, the main sewage pumps, and the blowers are all driven by steam turbines, and are all located in one building.

WHY STEAM TURBINES WERE USED FOR MAIN DRIVES

There were two reasons for using steam turbines for main drives instead of motor drives operating on purchased power. First, most of the elements of a steam generating plant, both in equipment and personnel, are required for the scheme of sludge disposal adopted, so that the additional annual cost of generating steam for power purposes is largely represented by the fixed charges on the small amount of additional equipment required plus the cost of additional coal burned. Second, there is sufficient fall between the final tanks and the Drainage Canal so that plant effluent from the south battery is available for condenser cooling water without additional pumping. The combined effect of these two factors is a very low power cost, even when compared with the very low rate at which the Sanitary District purchases electrical energy. For condensing purposes, the plant effluent is first screened through traveling screens of $^3/_{16}$-in. mesh and then dosed intermittently with chlorine on an automatically controlled cycle to eliminate organic growths and slime in the condenser tubes.

The four main sewage pumps each have a capacity of 300 cu ft per sec against a total dynamic head of 54 ft. They are preceded by mechanically cleaned bar screens with 2-in. clear openings. Three centrifugal blowers are provided for supplying air for aeration of the sewage, each with a capacity of 60,000 to 70,000 cu ft of free air per min against a gage pressure of approximately $7^3/_4$ lb per sq in. The intake air to blowers passes through three self-cleaning air filters of the oil-coated type, each with a capacity of 75,000 cu ft of free air per min.

The sewage pumps discharge into a common chamber, from which two influent channels lead to the preliminary settling tanks. To guard against accidental overtopping of the discharge chamber walls if tank gates are closed, siphon overflows leading to the effluent conduit and capable of passing the entire discharge of all pumps are provided. Owing to the presence of large quantities of wastes from the stockyards and packing houses, unusually large quantities of grease and other scum-forming materials may occur in the sewage. Hence the pump discharge chamber and influent channels to the preliminary tanks are made over-size and are provided with aeration devices to float as much grease as possible for removal in the preliminary tanks. The average detention in these aerated channels at a flow of 400 mgd is approximately 11 min.

AS FAR AS POSSIBLE, ALL FUNCTIONS REQUIRING HOUSING ARE LOCATED UNDER ONE ROOF
Here Is Shown the Pump and Blower House and Sludge Disposal Building; Coal and Ash Handling Facilities to Right

There are 12 preliminary settling tanks grouped into two batteries of six tanks each, in which grit as well as the coarser organic solids will be deposited, since separate grit chambers have not been installed. With the process of sludge handling adopted, separation of the grit may not be required, although space has been left so that grit chambers can be added later, if found necessary. The tanks in each battery are arranged in rows of three, on each side of an operating gallery surmounted by small superstructures for each opposite pair of tanks. The operating galleries and superstructures house the various pipe lines and control equipment for operation of the tanks. Each tank unit is 101 ft wide, 103 ft 6 in. long, and has an average water depth of 11 ft. The detention period at a flow of 400 mgd is 34 min. The deposited sludge is moved toward the inlet end by 6 parallel rows of motor-driven flight conveyors, the flights returning at the surface to move scum toward the effluent end. Two cross-conveyors at the influent end of each unit move the sludge to a central sump, from which an air lift pumps it to a discharge box, emptying into a pipe line leading to

the concentration tanks. A very thin sludge (99.7 per cent moisture under average conditions) will be drawn continuously at constant rate regardless of fluctuations in sewage flow and deposition of solids, the rate of draw-off being controlled by the air lifts. By this method,

AERATED INFLUENT CHANNELS AND PRELIMINARY SETTLING TANKS

constant velocity in the discharge lines will be maintained, which should minimize clogging. Scum is collected by a motor-driven cross-collector at the effluent end of each tank, operated as necessary, and removed in tipping bucket devices regulated to admit enough sewage to carry the scum through a pipe line to the point of disposal.

The effluent from the preliminary tanks flows through twin conduits to a gate and meter chamber, where the flow is divided and measured through venturi meters for distribution to the two aeration batteries. Ahead of the gate chamber, a second siphon overflow in the twin conduit may by-pass to the plant effluent conduit flows from the preliminary tanks exceeding the capacity of the aeration tanks and final settling tanks. The settled sewage destined for treatment receives the return sludge and is thoroughly mixed in aerated channels (one for each battery) 454 ft long, thence passing to the feed-channels for the two aeration batteries.

Each of these batteries consists of 8 aeration tanks and 16 final settling tanks, located on opposite sides of a long operating gallery. The tanks are so arranged that groups of two aeration tanks and four settling tanks form self-contained units independently controlled. A balancing channel, however, is also provided connecting the feeds to all settling tanks to maintain proper distribution in case individual tank units are out of service for repairs. Each aeration tank is 434 ft long and 134.75 ft wide, with a water depth of 15 ft over the air diffuser plates. At an average sewage flow of 400 mgd and a sludge return of 20 per cent of the sewage flow, the detention period in the aeration tanks is 5 hours. Tests indicate that from 0.4 to 0.5 cu ft of air per gal of sewage will be needed for aeration. Each unit is divided by three interior walls into four bays forming one continuous channel 1,736 ft long and 32.75 ft wide. Each bay contains two continuous rows of diffuser plates at the bottom and along one side, giving a plate ratio of 1 to 20.2. Each half of the tank is fed with air through a metered main, so that the inlet and outlet halves can be supplied with air at different rates if desired.

AIR LIFTS USED FOR RETURNING SLUDGE

The final tanks are of the circular, center-feed, peripheral overflow type, each 126 ft in diameter, with a side-wall water depth of 11 ft. At an average sewage flow of 400 mgd, the mixed liquor rate is 1,200 gal per sq ft of tank surface per day. Sludge is swept to a sump at the center of each tank by rotating motor-driven four-arm mechanisms, carrying plows suitably pitched.

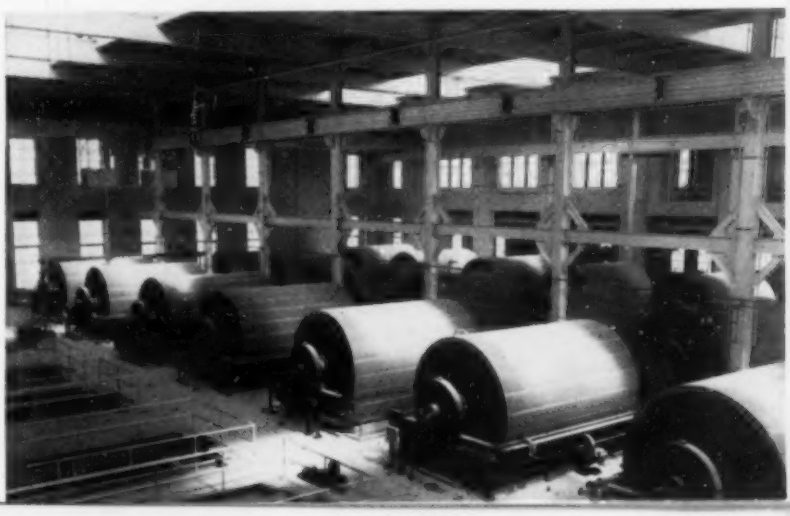
Among large-scale works, the Southwest Works is unique in the use of air lifts in place of pumps for creating the necessary head for returning sludge. This procedure was adopted for three reasons: (1) the difficulty of conducting into and out of a common pumping station the very large conduits that would otherwise be required; (2) the difficulty of securing efficient pumps of proper characteristics; and (3) the lower cost of the air-lift installation. When this method of sludge return was first considered, no data were available on air lifts for such very large quantities and low heads. Tests were made on lifts varying in diameter from 8 to 18 in., at heads up to 5 ft. As a result, two lifts, each 16 in. in diameter, were provided for each settling tank, one for ordinary flows, and the second to provide additional capacity for peak loads at a maximum rate of 45 per cent of average sewage flow.

REDUCING PEAK LOAD ON SLUDGE-HANDLING EQUIPMENT

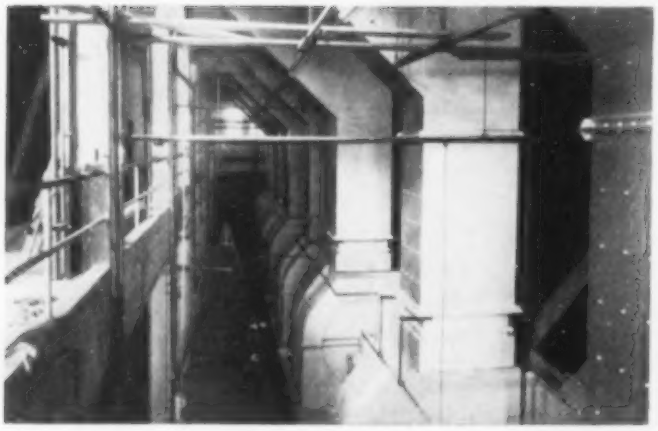
The exceptionally large pumping capacity for returning sludge results from the system of sludge disposal used. The operating records of the North Side Works indicate that incoming peak solids loads of 3 to 4 times the average might occur. To provide sludge-handling equipment for such large overloads appeared prohibitive. Study of the rates and frequencies of peak solids conditions showed that by building up storage of excess solids in the aeration system at such times to the extent of about 1,000 ppm, and later drawing down the stored solids when conditions return to normal, sludge-handling equipment only one-third greater than required for average conditions should suffice. The sludge disposal equipment is actually built on this basis, with three units for ordinary service and a fourth for peak-load conditions. The large pumping capacity for sludge return is needed to circulate the excess solids during storage periods. Further, waste solids can also be conveyed to the West Side Imhoff tanks, should operating conditions require.

In the outfall of the North Side Works, there are several drops, totaling about 18 ft. These induce a pickup of dissolved oxygen ranging from 5 to 8 ppm, of which a large portion is retained to aid in balancing the residual B.O.D. in the effluent. As the effluent from the north battery at the Southwest Works is not required for condensing purposes, the same principle is applied here, the excess fall to the main channel being concentrated in an effluent aerator. This consists of a double overflow weir, about 220 ft long, built over the effluent conduit.

Sludge wasted from the preliminary and final settling tanks is added to the sludge from the North Side Works and delivered to the sludge concentration tanks east of



ON THESE VACUUM FILTERS THE SLUDGE WILL BE PARTIALLY DEWATERED BEFORE PASSING TO THE FINAL DRYING UNITS



Cage-Type Drying Mills in Sludge Disposal Building

the sludge disposal building, at the Southwest Works. The average daily dry solids load is estimated at 375 tons. The concentration tanks are planned to dewater the sludge as far as practicable by settling, prior to vacuum filtration. For average conditions, the retention period in the concentration tanks is about 4 hours. As the sewage at the Southwest Works has been screened only through rack screens, with 2-in. clear openings, the waste sludge is passed through three units of mechanically cleaned bar screens with $^1/_2$-in. clear openings before entering the concentration tanks, to prevent clogging in pumps and pipe lines on the filter feed system. Six rectangular tanks are arranged symmetrically on both sides of an operating gallery, each tank unit being 46.75 ft wide, 70 ft long, and having an average water depth of 14.5 ft. In each unit concentrated sludge is moved to the gallery end by conveyor equipment similar to that used in the preliminary tanks and then pumped through duplicate force mains to the sludge disposal building. Each tank is provided with a separate variable-speed pumping unit to control the withdrawal of concentrated sludge.

DRYING AND BURNING THE SLUDGE

The handling of the sludge involves partial dewatering on vacuum filters, "flash" drying, and burning in suspension in furnaces where pulverized coal is also being burned for the generation of steam for power purposes. The combined ash is to be disposed of by dumping on Sanitary District lands along the Ship Canal. If desired, heat-dried sludge can be withdrawn from the system and sold for fertilizer. In such case, about one-half ton of additional fuel is required per ton of dry sludge removed.

The sludge is first coagulated by the addition of ferric chloride in the amount of approximately 5 per cent of the dry weight of the sewage solids. It is then dewatered on vacuum filters to reduce the moisture content to between 78 and 82 per cent. Twenty-four filters, each with a filtering area of 570 sq ft, are provided. The average filter output is estimated at about 3 lb of dry solids per sq ft per hour. The filter cake is then broken down and evenly distributed on a collecting conveyor belt which feeds mixing devices attached to each drying unit, in which sufficient previously dried sludge is added to produce a mixture with a moisture content between 40 and 50 per cent.

This friable mixture is introduced into a closed-circuit duct system circulating superheated water vapor and other gases at 1,100 F and atmospheric pressure, and passes to a squirrel-cage disintegrating mill and thence through a drying duct to a cyclone separator, where the dry solids drop out of the vapor stream and are removed to a surge bin. From this bin the required quantity is taken for mixing with the partially dewatered filter cake, and the remainder is blown into the furnace in an air suspension, or diverted for fertilizer. The separated vapor stream is returned through a heat exchanger or reheater, which raises the temperature again to 1,100 F.

Each of the eight drying circuits is designed to evaporate 20,000 lb of water per hr. The excess sludge vapor thus produced is discharged through relief ducts into the furnaces; thus all odors are destroyed in passing through the combustion zone. The dry sludge in Chicago averages about 7,000 Btu per lb of dry solids. This supplies sufficient heat to evaporate the moisture in the sludge when the filter cake contains 80 per cent moisture. At lower moisture content, some heat may be available for generating steam. At higher moisture content, additional heat will be supplied by burning more coal. Further adjustment may be required because of fluctuations in the content of volatile or combustible matter in the sludge, as this may vary from 40 to 80 per cent of the dry solids, averaging about 60 per cent.

Steam for plant operation will be generated by four water-tube boilers, having a maximum capacity of 110,000 lb of steam per hour at a pressure of 425 lb per sq in. and a final temperature of 735 F. Coal can be received by rail or water, then crushed and elevated to bunkers from which it feeds by gravity to the pulverizers at each furnace. About 20 per cent of the ash is a granular material falling into the ash hopper, from which it will be sluiced to an ash lagoon, or to a dewatering ash hopper. The remainder, or fly ash, is intercepted by electrical precipitators, in two units, each capable of handling 180,000 cu ft of gas per minute, before the flue gas passes into the stack. Mechanical heat exchangers on the boiler breechings extract heat for preheating the combustion air to about 400 F.

DESIGN MAKES FOR EASE AND ECONOMY IN OPERATION

Simplicity, compactness, and economy (both from a construction as well as an operating standpoint) are the outstanding features in the design of the Southwest Works. As far as possible, all functions requiring housing are located under one roof. Other plant structures are connected to the central building by a system of underground service tunnels. A very complete system of metering is provided, with recording and integrating instruments for main sewage, air, and sludge flows. All superfluous equipment has been carefully eliminated, in the light of operating experience on a large scale at the North Side and Calumet works. In the main building, office and laboratory facilities are located in a tower, and shops, storage, and garage facilities are provided in the basement.

The Southwest Works was built in cooperation with the Federal Public Works Administration (PWA), as part of a large program for the construction of intercepting sewers and sewage treatment works.

Aeration and Final Settling Tanks
This Installation May Ultimately Be Tripled

Protective Standards of the Federal Housing Administration

By Seward H. Mott

Director, Land Planning Division, Federal Housing Administration, Washington, D.C.

Many will welcome this clear statement of the work of the Federal Housing Administration—often confused with the U. S. Housing Authority, an entirely separate agency. Mr. Mott explains the function of the FHA as a mortgage insurance company and details the conditions under which it executes mortgages. He also explains the important rôle played by zoning and restrictive covenants in making sites eligible for acceptance. This paper was on the program of the City Planning Division at the Society's 1939 Annual Meeting.

The preamble to the National Housing Act, under which the Federal Housing Administration (FHA) is organized, states that it is an act to encourage improvement in housing standards and conditions and to provide a system of mutual mortgage insurance. In contrast to the U. S. Housing Authority, the FHA loans no money. It is in effect a mortgage insurance company to which the mortgagor pays premiums from $^1/_4$ to $^1/_2$ per cent yearly. In this way a fund of some $24,000,000 has been built up to meet losses, which to date have been only about $160,000. Financial institutions are insured against loss on loans approved by the FHA. Certain types of these loans can be had for as much as 90 per cent of the value as appraised by the FHA, and for a period of as long as 25 years. These loans are amortized monthly, the payments including taxes, interest, premium, and reduction of principal.

To safely insure such high-percentage loans, the act requires that all projects for which mortgages are executed be economically sound. In ascertaining such soundness, the FHA gives particular consideration to (1) the credit and financial reputation of the borrower, (2) the soundness and durability of the structure, and (3) the character of the neighborhood. The effect of this last factor is of vital importance, since neighborhoods in a growing community are subject to constant change. The fine old residential area of a decade or more ago, when unprotected, becomes the boarding-house and small business district of today. To solve this problem, the FHA has established standards which eligible neighborhoods must meet.

Factors Affecting Long-Time Residential Values

The "life expectancy" of a residential property in so far as its location is concerned is determined by an analysis of the following factors:

1. The economic stability of the city of which it is a part
2. Protection provided by zoning, and restrictive covenants or natural physical protection against undesirable encroachments
3. Freedom from the hazards of flood, earthquake, subsidence, or other acts of Providence
4. Adequacy and accessibility of schools, parks, playgrounds, churches, stores, and places of employment
5. Adequacy and cost of public transportation
6. Sufficiency of such utilities as water, sewers, electricity, gas, and such improvements as roads and walks
7. The level of taxes and special assessments
8. Appeal of the neighborhood to possible buyers

Each of these factors is carefully weighed and rated and the absence of any one of these basic qualifications will result in rejection of the site for insurance purposes. High ratings on all factors assure high-percentage loans.

A careful analysis of the destructive factors affecting residential areas shows conclusively that they can, to a great extent, be controlled by carefully drawn and conscientiously administered planning and zoning regulations and restrictive covenants.

The attitude of the FHA with regard to zoning is set forth as follows in its "Manual of Instructions to the Underwriting Staff":

"One of the best artificial means of providing protection from adverse influences is through the medium of appropriate and well-drawn zoning ordinances. If the provisions of an ordinance have been well worded and drawn from a thorough knowledge of existing and probable future conditions in the city, and if the ordinance receives the backing of public approval, an excellent basis for protection from adverse influences exists .... [However] even when an ordinance is ably drafted, investigation must be made to determine whether infractions of the zoning law are permitted .... Greater importance is attached to zoning protection in and near large metropolitan centers than in places having smaller populations and less rapid rates of growth. Absence of zoning may be a proper basis for rejection in the former case, but would not necessarily cause rejection in the latter case."

A rather interesting example of the way this requirement works was recently shown in a large city in the far South. It has a zoning ordinance which for a time was well administered. Suddenly it was found that the Zoning Board of Appeals was making exceptions in residential areas, and the FHA at once tightened up on the terms and percentages of loans. There was an immediate reaction from the Real Estate Board and the Chamber of Commerce, both of which presented objections to the FHA. The FHA explained its position—that it could not make full-percentage loans when no protection was afforded by the zoning ordinance because of exceptions. A committee was then formed to meet with the city officials, and as a result the condition was promptly corrected.

In considering the protection provided by a zoning ordinance, the FHA is chiefly concerned with those clauses which affect residential areas and their location. In many cases the zone boundary runs down the center of the street, instead of being in the middle of the block. It has been our experience that the value of residences facing non-conforming uses is definitely decreased, and we therefore encourage the same type of use on both sides of every street.

We also find that many zoning ordinances do not provide for the garden type of apartment. By this I mean two- to three-story apartments having a land coverage of not more than 25 per cent and a density of not over 20 families per acre. Apartments of this kind provide excellent housing, and should be permitted in outlying districts adjacent to good residential areas.

In many communities, apartments are still thought of in terms of the high, dense type and consequently are relegated to blighted areas or to buffer strips with the hope that an increase in land values will result and serve to retard the encroachment of blight upon the better residential neighborhoods. Generally this fails of its purpose, especially where the demand is light and the transition slow. Merely permitting a more intensive use without adding protective measures to insure continuing residential desirability, is in itself an ultimate source of blight. On the other hand, zoning regulations that will foster the garden type of apartment will tend to create sound values that will resist blight.

The FHA encourages zoning that provides for neighborhood "park and shop" centers on suitable sites and at appropriate intervals. It does all it can to discourage commercial strip zoning along both sides of important highways. It is apparent that when local stores are grouped with architectural unity and with parking space provided, they are an asset to a neighborhood. The contrary is true when they are built haphazardly along a highway with thousands of feet of vacant frontage for which there is no need and which quickly becomes unsightly.

We also advocate comprehensive county zoning and cooperation of municipal zoning units on a regional basis. We frequently endeavor to get several small communities to work out their city plan and zoning ordinances together, and perhaps retain one technician to take care of the problems of the group. This eliminates conflicts where city lines meet.

It should be made clear that the FHA insures many loans in built-up areas not protected by zoning ordinances. There is a surprisingly large number of communities that have no zoning ordinances or restrictive covenants of any kind. In such areas we seldom are able to insure full-percentage long-term loans.

SPECIAL REVIEWS REQUIRED FOR UNDEVELOPED AREAS

In new subdivisions or in undeveloped areas either old or new, which are in such a state of development and unified control as to make practicable major changes in layout, restrictions, or other features, a special analysis and review must be made by the Land Planning Division before loans can be insured. This review sets up certain requirements for FHA participation, as follows:

1. Convincing evidence of continuing demand for housing in the proposed price range
2. Appropriate surroundings and topography
3. Accessibility to schools, employment, shopping and recreational centers
4. Suitable utility and street improvements
5. Reasonable taxes and assessments
6. Adequate zoning and restrictive covenants
7. Conformity to planning regulations and sound principles of design.

In outlying areas without zoning protection, we request the sponsor to make every effort to have adjoining property owners restrict land use in a manner comparable to his own. Failing this, we endeavor to adjust the plot plan so that the minimum number of lots face uncontrolled property, and we frequently recommend that objectionable views be screened by appropriate planting.

Where there is considerable residential frontage along fast-traffic arteries, we endeavor to secure protection by means of planting panels about 20 ft wide, and a supplementary 16-ft roadway behind them to serve the residences. In this way the hazard of driving directly into fast traffic is eliminated and the objectionable features inherent in facing noisy, fast traffic are largely avoided.

The standards of the FHA with respect to subdivision design are quite simple and much less detailed than the average ordinance. They are as follows: "The subdivision plan must be suitable for the site and appropriate for the use intended. Subdivision layouts are not acceptable if they adversely affect the appeal and marketability of the properties or unnecessarily add to the cost of making the land ready for use." The method of subdivision review by experienced planners on the staff of the Land Planning Division and the required standards insure a quality of design that is seldom equalled by local planning boards. Where subdivisions are submitted in which many sites have been sold or improvements installed so that major changes in plan are impracticable, loans may be rejected or made on a low-percentage basis.

RESTRICTIVE COVENANTS TO MAINTAIN VALUES

Even in areas that are well controlled by zoning we require the additional protection of restrictive covenants placed as blanket encumbrances against the entire tract. Where zoning controls the type of development, restrictive covenants may be said to control the character of development. They should cover the designation of definite areas for specific uses such as business, homes, and parks, and the intensity of such uses. These classifications may be more restrictive and more detailed than any applicable zoning ordinance.

We require that provision be made for adequate light and air through generous front, rear, and side yard setbacks. Regimentation of structures may be avoided through variations in the front setback lines. Resubdivision and its attendant land crowding is eliminated by the establishment of minimum lot sizes and by permitting only one dwelling on each lot. The use of temporary residences, the moving on of old structures, and nuisance uses are prevented. One of the most important covenants is that which deals with architectural control and minimum building sizes. This protection is best obtained through appointment or election of a committee of resident property owners, whose duty it is to pass upon the design and location of structures within the subdivision.

We recommend that the size of the structure be controlled by a minimum ground-floor area stipulation, rather than solely by a minimum building cost. There are many good subdivisions where development has been arrested by too high cost limitations placed on the property some years ago when economic conditions were on a different level.

Subdivisions must conform to local regulations and to the recommendations of local planning boards, and sales must be made from recorded plats, rather than by metes and bounds.

The FHA makes no original subdivision plans but, when given an opportunity, does suggest changes. On the sketch showing suggested revisions is stamped the following statement: "This study is simply a suggestion to assist the developer in planning his property. It should not be considered a complete or final plan. It is recommended that a competent subdivision designer be retained to work out the detailed plans and assist in other subdivision problems." In its publications and in its conferences with real estate operators and builders the FHA continually stresses the need of retaining qualified subdivision designers.

Our offices are instructed to cooperate fully with the planning and zoning authorities and with other organizations interested in better community development, for only in this way can our objective—"The creation of stable, attractive neighborhoods"—be attained.

The Structural Significance of Stress

From an Address Before the Lehigh Section

By Bruce Johnston, Assoc. M. Am. Soc. C.E.

Assistant Professor of Civil Engineering, and Assistant Director Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

What is the relation between stresses in a ductile material and the yielding of the material? What is the relation between the yielding of the material and the failure of an engineering structure? What is the true concept of "factor of safety"? What factors influence the "useful limit" of a structure under static load? Finally, what is the relation between these factors and the everyday problem of design? Professor Johnston, in correlating what is known about these questions from the viewpoint of their application to structural engineering practice, makes a helpful contribution to the interpretation of current structural research.

The structural engineer is confronted today with questions involving revision of working stresses, selection of factors of safety, and modification of familiar methods of analysis and design. It is the purpose of this article to correlate some of the factors that are fundamental to a study of these questions.

Primary attention will be given to the behavior of structural steel members loaded statically. The interrelation between the following subjects will be discussed: (1) stress analyses and the state of stress, (2) yielding of materials and yielding of structures, (3) experimental research and the load history of a structure, and (4) working stresses and the factor of safety in design.

A simple case will illustrate the relations under discussion. A steel beam may be designed for a working stress of 20,000 lb per sq in. on the basis of the usual "beam formula," that is, stress equals moment divided by section modulus. The "factor of safety" in this case might be thought to be equal to the yield-point stress of the material divided by the working stress of 20,000 lb per sq in. In the case of structural steel with a yield point of 33,000 lb per sq in., the computed factor of safety would thus be 1.65. On the other hand, a more exact analysis of local stresses under bearing blocks or in fillets might show these stresses to be already above the yield point of the material. Naturally this does not mean that there is no factor of safety. Actually, if the beam in question is put in a testing machine and loaded it will not yield as a structural unit until the computed stress by the beam formula is well above the yield-point stress of 33,000 lb per sq in. The real factor of safety, defined as the load ratio between the "general yielding" and the design load, will be nearer 2 than 1.65.

Stress Analyses and the State of Stress

In studying the fundamental relations between the state of stress and the physical properties of a ductile material, it is essential to consider the three-dimensional character of stress. Imagine a very small cube cut from the interior of any member under load, as shown in Fig. 1 (*a*). The cube is imagined to be microscopic in size so that the resultant stress may be considered as uniform over each plane face. In general there may be a different resultant stress on each of these three faces, with equal and oppositely directed stresses on the three faces hidden from view. Each of these three stresses may be resolved into three components parallel with the x, y, and z axes. The determination of these nine components of stress at every point in a structural member in accordance with the conditions of static equilibrium, the equations of continuity or compatability, and the boundary conditions, constitutes a stress analysis for a mathematically idealized material. Solutions of typical problems may be found in standard treatises on the theory of elasticity. It is also possible to determine by photoelasticity the stress distribution in a Bakelite model made to simulate the actual structure. Photoelastic studies have usually been two-dimensional but the extension to the three-dimensional problem recently has been made possible.[1] Stresses on the surface of actual structures may be explored by means of strain-rosettes. The stresses in the interior of dams have been determined by casting electric telemeters or electric strain gages into the concrete. Various analogies are available for experimentally determining stresses for certain special problems.

It will be assumed here that a stress analysis has been obtained by one of the foregoing methods. What is the significance of this analysis to the engineer? To answer this question it is necessary to begin by relating the state of stress to the initial elastic failure of the material.

As a preliminary to the study of the yielding of materials, it is convenient to simplify the three-dimensional state of stress illustrated in Fig. 1 (*a*). It is always possible to determine a new orientation of the direction of the axes of the cube such that the shearing components will vanish, leaving only the normal stresses σ_1, σ_2, and σ_3, as shown in Fig. 1 (*b*). One of these principal stresses will be a maximum stress and another will be a minimum.

Now consider an octahedron constructed by connecting the centers of each face of a cube which is oriented in the principal directions, as shown in Fig. 2. It may be shown[2] that no matter what state of stress exists at a particular location, the normal stresses on each face of the octahedron are identical and equal to:

$$\sigma_n = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

Likewise, the magnitudes of the shear stress on each octahedral face are identical and equal to:

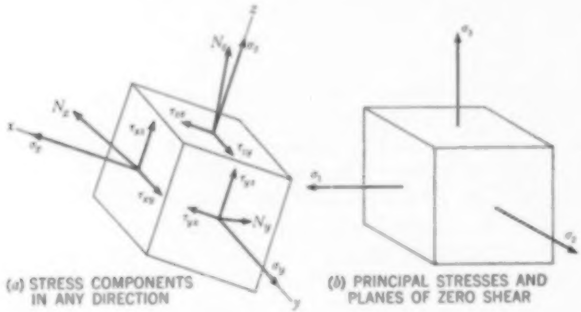
$$\tau_n = \frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2}$$

Any state of stress is thus reduced to a very simple concept which has particular significance when the initial plastic yielding of a ductile material is to be determined.

Relation Between State of Stress and Physical Properties and Failure of Materials

Concepts regarding all the physical properties of a material are usually based on the behavior of the material under one-directional loading in a tension or compression test. In the actual structure the stress is usually not one-directional, and the physical properties of the material as usually conceived may no longer hold.

The failure of the material will now be considered, and two types of failure will be distinguished. In a simple tension test, if the material elongates considerably after initial yielding before it finally fractures it is said to be a ductile material. The initial plastic yielding will be termed elastic failure. A material that breaks suddenly in a simple tension test, with little or no elongation or reduction in area, is termed a brittle material. Nadai[3] and others have shown, however, that ductility and brittleness as exhibited by the type of fracture and the ability to withstand permanent elongation without fracture depend not only on the nature of the material but on the state of stress as well.



(a) STRESS COMPONENTS IN ANY DIRECTION (b) PRINCIPAL STRESSES AND PLANES OF ZERO SHEAR

FIG. 1. GENERAL STATE OF STRESS

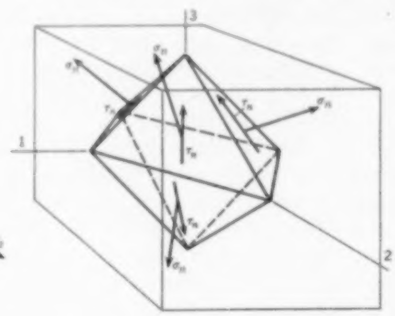


FIG. 2. THE OCTAHEDRAL STRESSES

The theory of elastic failure which to date gives the best evaluation of initial plastic yielding in a ductile material under a state of combined stress is usually called the von Mises–Hencky theory. It is also called the shear-strain energy theory of failure but may be defined without reference to strain energy, as follows:[2] "Plastic yielding in a ductile material will result when the octahedral shear stress reaches a certain limiting value." Let this limit of octahedral shear stress be denoted by τ_{oy}. In a tension test with tensile stress σ_1, acting in one direction only, the octahedral shearing stress $\tau_o = 0.47\sigma_1$. If the yield-point stress, σ_{yp}, is 33,000 lb per sq in., one may therefore state that elastic failure or plastic yielding in mild structural steel will occur when the octahedral shear stress $\tau_{oy} = 0.47 \times 33{,}000 = 15{,}500$ lb per sq in. It should be kept in mind that the octahedral shear stress is usually not the maximum shear stress.

In Fig. 3 are shown the stress-strain relations and initial yielding of structural steel as evaluated by this theory for three stress combinations in addition to simple tension. For the state of uniform all-round tension, the load at initial plastic yielding is theoretically infinite. It is experimentally impractical to produce a uniform all-round tension, but the conclusion is not inconsistent with the theory because in such a state of stress we may expect no plastic yielding since no shearing stress is present. Under a state of all-round tension a "ductile" material would exhibit the characteristics of a brittle material. Brittle failures of this type may sometimes be caused by zones of three-dimensional tension set up as internal stresses due to cooling.

The yielding of material under a state of uniform combined stress has been discussed. What is the relation between the yielding of the material and the general yielding of the structural member? A tension member of uniform cross-section (such as an eyebar), and a hollow tube in torsion, are two of the rare instances in which a state of uniform stress critically affects an entire structural member. In such cases one may expect the member to yield and fail at loads corresponding to the yield-point stress and ultimate strength of the material. In other structures or structural units—such as beams, columns, rigid frames, and floor slabs—it is well known that general yielding of the structure does not coincide with the load at which the material at some particular point passes the yield point.

THE LOAD HISTORY OF THE STRUCTURE

The load history of a material or a structure is the complete record of its behavior from initial load to final failure. The load history of the material is usually recorded by the stress-strain graph of a standard tension test together with data regarding elongation, reduction in area, and so forth. The load history of the structure as a whole may be studied by plotting the load against a deflection or deformation that is associated with the over-all behavior of the structure, as shown in Fig. 4. In a few special instances, such as the bending of a beam or the twisting of a circular rod, the load history of the structure may be calculated analytically from that of the material by the theory of plasticity.[3] Usually the structure is studied experimentally in the testing machine of the laboratory or under dead-weight loading in the field. On this subject Hardy Cross states:[4] "The interpretation of stress analysis makes absolutely necessary a clear idea of the action of the structural part up to the stage at which rupture is conceivable."

The experimentally determined load histories of a standard tension test and the large-scale test of a structural unit have qualitative similarities. The following subdivisions may be made for each case:

1. Initial readjustments of grips or bearings at low load
2. The elastic range, wherein load is proportional to deformation
3. The proportional limit
4. The yielding of bar or structure
5. Maximum ultimate load
6. Final fracture or complete failure.

A typical load deformation curve, such as is shown in Fig. 4, gives a graphical record of these stages in the load history of the structure.

In the elastic range the stress distribution on the surface of the structure may be determined experimentally by strain-rosette readings.[5,6] The proportional limit may be determined approximately as the point at which the load deformation graph deviates from a straight line. Between the proportional limit

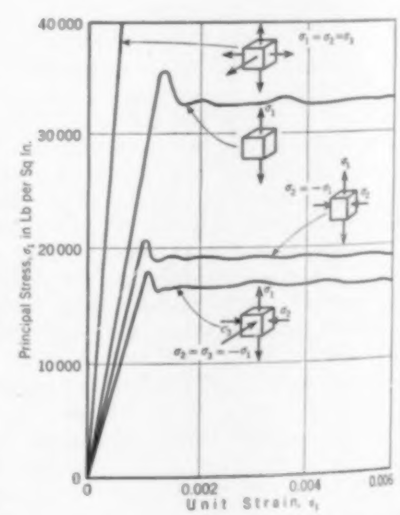


FIG. 3. RELATION BETWEEN MAXIMUM PRINCIPAL STRESS AND CORRESPONDING STRAIN FOR STRUCTURAL STEEL UNDER DIFFERENT STRESS COMBINATIONS

and the general yielding of a steel structure, local yielding may be noted by the flaking off of mill scale along well-defined lines, which indicate the intersection between planes of maximum shear and the surface of the structure.

The general yielding of the structure is particularly important as it is the reference point of a real factor of safety. It usually represents a limit beyond which the structure will no longer usefully serve its original purpose. Methods for determining this "useful limit" correspond in some cases to those used in determining the yield point of a material in a simple tension test. Four methods will be outlined here, the useful limit of the structure in each case being the load at which:

1. A limiting amount of permanent deformation has taken place.
2. The slope of the load-deformation curve is smaller than the slope of the original tangent by an arbitrarily determined ratio.
3. The point at which the load-deformation curve has the highest rate of change of curvature. This may be determined approximately by the construction shown in Fig. 4.
4. A point which depends not only on the initial behavior but on the ultimate strength as well, and which is obtained by the other construction shown in Fig. 4.

All these methods have advantages and disadvantages. Method 1 is the simplest and gives a well-defined value but depends on specifying an arbitrary allowable def-

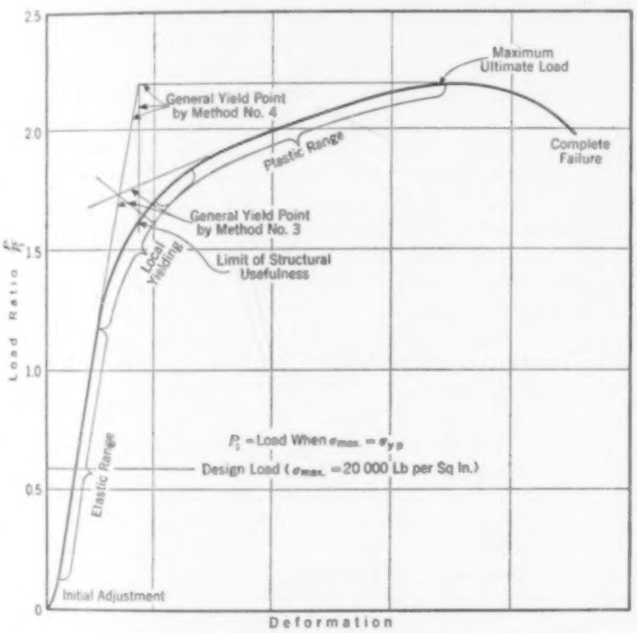


FIG. 4. TYPICAL LOAD-DEFORMATION CURVE FOR A STRUCTURE

ormation. Method 2 depends on specifying an arbitrary slope and also has the disadvantage of not giving a well-defined point if the material or structure yields slowly. Method 2 is quite satisfactory if there is a sharp break in the curve. Method 3 is ideal in obtaining very nearly the sharpest break in the curve and is especially adapted to types of failure in which there is a secondary region above the yield point where the deformation is nearly proportional to the strain. The last method, No. 4, has the advantage of giving some weight to the ultimate strength of the material or structure. Methods 3 and 4 have recently been tried out on a research project at the Fritz Engineering Laboratory of Lehigh University[7] and have the advantage that they do not depend on any arbitrary slopes or deformations. Method

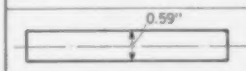
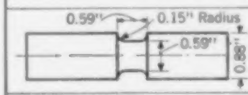
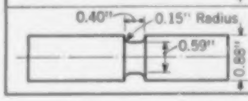
TENSILE SPECIMEN	UNANNEALED			ANNEALED		
	First Yielding	Upper Y. P.	Lower Y. P.	First Yielding	Upper Y. P.	Lower Y. P.
	Kips per Square Inch					
	—	48.2	42.2	—	43.2	40.2
	43.4	45.4	44.2	36.6	42.2	41.8
	42.4	52.2	51.4	38.1	50.1	49.3

FIG. 5. TENSION TESTS OF GROOVED BARS (NADAI)

4 is the most definite and least arbitrary of all but may define a limit of structural usefulness somewhat above the actual yield point in cases where the ultimate strength is high. In any actual research program, satisfactory results should be obtained by the consistent use of that method which is judged to be best suited to the particular program in question.

FACTORS INFLUENCING LOAD HISTORY OF A STRUCTURE

The load history and useful limit of a structure in a static load test will be affected by one or more of the following factors: (1) the physical properties of the material used, (2) the state of stress in various parts of the structure, (3) the degree to which maximum stresses are localized and the structural importance of this location, and (4) the stability of the structure and its component parts. The following specific illustrations will demonstrate the influence of these factors.

Stress Concentrations. It is well known that stress concentrations due to sharp reentry corners or holes in structural members cause high localized stresses. In Fig. 5 are shown the results of tension tests[8] on polished steel specimens all having the same net section. The apparent yield point has been raised in the grooved bars because local constriction is prevented by the shape and by surrounding low stressed areas. The nominal ultimate strength is also raised because necking down is prevented, since planes of slip cannot develop freely. The notched bars, however, would undergo much less deformation before fracture occurred, and the fracture would be characteristic of a brittle rather than a ductile material. These bars would be particularly poor in resisting impact loads or repeated stress.

Stress concentrations have a similar effect in bending. A steel bar having two circular grooves and dimensions as shown in Fig. 6, was loaded as a cantilever beam. The load-deformation diagram deviated almost imperceptibly from a straight line when the stress concentrations in the groove as determined photoelastically[8] reached the yield point of the material. But the beam did not yield as a structure until a load was reached which was 53 per cent above that calculated with the stress concentration effect included, or 28 per cent above the load calculated by ordinary beam theory, neglecting stress concentrations entirely. This higher strength is partly due to the low stressed areas on each side of the notch, and partly to the inherent reserve strength of a beam in bending, which will be discussed as the next topic.

Effect of Cross-Sectional Shape and Distribution of Load. The general yield, or "useful limit," of an elastically stable beam always occurs at a load higher than the computed load at which the material in the extreme fibers passes the yield-point stress of the material.[9] For beams of equal section modulus, loaded at the center with a single concentrated load, the increase in useful limit is nearly 100 per cent for a circular beam, over 50

per cent for a rectangular beam, and is between 15 and 40 per cent for I-beam sections of various proportions. Plastic yielding for beams loaded at the center commences at a very localized region at the top and bottom of the center of the beam. In the solid beam there is a large reserve of material, but in the I-beam section most of the effective material in the cross-section is immediately stressed above the yield point.

Beams loaded at the third-points, or uniformly, have slightly lower useful limits than corresponding beams with center loading. In the third-point loading all the extreme fibers between the two load points pass the yield point at the same moment, thereby affecting at equal stress a greater percentage of the material than in the case of center loading.

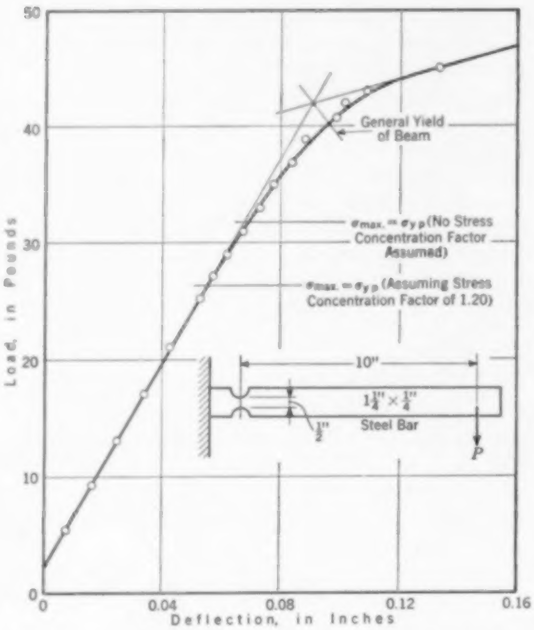


FIG. 6. RELATION BETWEEN LOAD AND DEFLECTION FOR CANTILEVER BEAM WITH GROOVE

The shape factor may be illustrated in torsion by comparing the moment-twist curves of a solid and hollow bar having the same polar moment of inertia. In the hollow round bar all the material is in a nearly uniform state of stress, and initial yielding of material results at once in the general yielding of the structure. In the solid bar there is a reserve elastic region after the outer fibers have exceeded their yield stress locally, and as a result the general yield strength is raised.

The influence of both shape and load factors may be stated in another more general way, that is, the useful limit or general yield point and the shape of the load-deformation curve after yielding depend on the relative percentage of material in which the yield point is exceeded and the rate at which this percentage changes. The most effective shapes in the elastic region are the least effective after the maximum stress passes the yield point. This is the natural result of putting as much material as possible in the regions of highest stress.

Other Factors. If the structure is statically indeterminate, other factors affect the load history of the structure. The complete yielding of one part of an indeterminate structure may still leave a stable structure. As an example consider a frame consisting of two columns and a horizontal beam attached to the columns by angle connections. After the connections start to yield, the load deformation curve will continue on a new slope, but the structure will still be quite safe as it will be in a state of transition from a continuous frame to a statically determinate system consisting of two columns and a simply supported beam between.

One example in the field of buckling will illustrate another possibility. If plate girders with vertical stiffeners were made with webs thinner than are now allowed, a web could be designed that would buckle at extremely low loads. Although the web would quickly become useless in compression, it would still be able to carry tensile forces from the top of one vertical stiffener to the bottom of the next. The intermediate stiffeners would serve as compression struts, and as a result the girder would behave as a truss instead of a beam. Aeronautical engineers consider this in designing wing girders.

Examples have been given to illustrate some of the many factors that affect the strength of a structure. What is really desired is a safe structure, and every type of structure behaves to a certain extent according to laws determined by its own peculiar characteristics. To repeat—the real factor of safety in any structure is not the ratio between calculated working stress and yield-point stress but rather the load ratio between working design load and load at the limit of structural usefulness. In connection with present design methods, the important problem in every case is to specify the correct allowable working stresses to give a safe load ratio. In some cases it may be necessary to specify also the manner or degree of precision with which the stresses are to be computed. This may be important because the calculated stresses may be based on an approximation which depends for its exactness entirely on the method of computation used.

In unusual design problems requiring special stress analyses by methods of elasticity or photoelasticity, consideration must be given not only to the magnitude of the maximum stresses but to their location, state of combination, and probable effect on the behavior of the structure as a whole. No definite working stresses can be specified in such cases.

The principal problem of the structural designer is that of proportioning a structure which will be both economical and safe, and which will give satisfactory service in these respects throughout its useful life. New types of structures, such as concrete shell domes, steel rigid frames, and welded structures of all kinds, are coming into use. These require experimental and theoretical research in order to evaluate the allowable loads and allowable computed stresses. The factors that have been discussed, as well as other practical questions such as expected corrosion, repetition of stress, expected life of structure, and hazard to human life—all have a bearing on the selection of the proper allowable unit stress to determine a load factor that will result in a safe and enduring structure.

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Economic Value of Traffic Engineering

Recent Studies Carry the Traffic Engineer Into the Related Fields of City and Regional Planning and Increase His Usefulness to the Community

By D. Grant Mickle

Associate Member American Society of Civil Engineers

Manager, Traffic and Planning Department, Jensen, Bowen and Farrell, Engineers, Ann Arbor, Mich.; Assistant Director, Michigan Highway Planning Survey

The field of traffic engineering is broadening immensely, and with it the point of view of the traffic engineer. Only a few years ago, it seemed that his chief function was merely to deal with disorganization and friction in traffic movement. But today he must recognize that his thoughts and methods of attack must include the fundamental causes of which traffic troubles are but symptoms. Important as are the problems with which traffic engineering has up to now been concerned, and valuable as are the contributions it has made to their solution, it is clearly only at the beginning of its usefulness.

Unfortunately, the widespread use of the automobile, both in urban and rural areas, could not be foreseen by the builders of our city streets and state trunk lines. The need for sound planning is generally recognized too late and, up to now, much of the work of the traffic engineer has been in making the best of these omissions of the past. Even in this limited field he has often been handicapped by the lethargy of officials in failing to recognize the importance of the traffic problem.

In a report of traffic conditions in Toledo, Ohio, ("Traffic Planning Report—Toledo, Ohio," Jensen, Bowen and Farrell, Engineers), it was shown that public expenditures for services designed to cope with traffic accident losses were not at all in proportion to the amount of such losses (Fig. 1). More than two dollars was allotted to the fire department for each dollar of actual fire loss, while only 34 cents was spent for traffic control and betterment services for each dollar of traffic damage loss. This example indicates a lack of official appreciation of the magnitude of traffic losses and a lag in the public effort to cope with them.

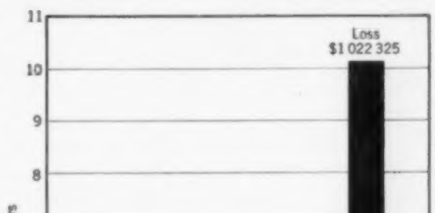


Fig. 1. Traffic and Fire Losses Compared to Expenditures in Toledo, Ohio

Two Dollars Are Spent for Fire Prevention for Every Dollar of Fire Loss, Yet Only 34 Cents Go for Traffic Accident Prevention for Every Dollar of Traffic Loss

Under even the most ideal conditions of street and highway capacity and pattern, planning the most effective use of these facilities is necessary. But traffic engineering includes also the determination of what these facilities should be. It is thus inseparably linked with city, state, and regional planning.

The fundamental factors to be considered in preparing a traffic plan are location and concentration of population, both in rural and urban areas; location of, and employment in industries, offices, and retail establishments; extent, kind, and location of recreational centers; location and arrangement of the thoroughfare system; and type of land use. The relation of these factors to one another, and their influence upon congestion and accidents, make up the traffic problem. Much of the data required by the traffic engineer is of a character usually prepared by a city plan commission. This emphasizes the close relationship between city planning and traffic engineering and the need for close cooperation between these two municipal activities.

In dealing with this problem the traffic engineer has been faced with a difficult situation. Traffic accidents constitute the spectacular phase of his work, and there has been and still is an insistent public demand that direct action be taken to stop them. Yet the real causes of accidents, as well as of other conflicts and congestion on the roadways, are usually of such a fundamental nature that their correction is costly, and the specific benefits from any particular expenditure are often intangible. However, if traffic engineering is to deserve the full esteem we all wish for it, we must meet our responsibilities, as regards both accidents and traffic efficiency, by attacking these basic causes. It is possible to adopt this policy, confident that the same savings will result from it as are realized when traffic engineering is applied to the more superficial problems of traffic regulation.

To quote from the 1936 report of the Committee on Street Traffic of the American Transit Association: "The benefits of improved traffic facilities may seem intangible and the investment non-liquidating to the agency making the expenditure. A real saving accrues to both the community as a whole and its individual citizens, nevertheless, if improvements are well planned. Eventually the cost is repaid in enhanced property values, in greater consumption of taxable goods, and in the economy of properly designed and executed city planning."

A survey of street traffic conditions in Detroit, "Street Traffic—Detroit" (Michigan State Highway Department), illustrates the various phases of high original cost and eventual effectiveness and economy which are involved in a thoroughgoing attack on basic factors of traffic congestion and accident occurrence in a great city.

Studies of traffic volumes and of the origin and destination of traffic disclosed that the downtown streets carried not only vehicles bound for destinations within the district, but also thousands of other cars which passed through the district to distant destinations because adequate cross-town arteries were lacking. Investigations of parking conditions, require-

ments, and habits indicated that desirably located off-street facilities were inadequate and that all of the legal and much of the illegal curb space was occupied. Not only were terminal facilities insufficient but the radial arterial streets were overcrowded for several miles away from the central business district. It was estimated that the driving public was paying $10,000,000 annually for congestion and delay alone.

The results of these unfavorable conditions upon business were revealed when comparisons of traffic counts over a period of 12 years showed that the number of people coming into the district had actually declined almost 4 per cent, although passenger car volumes had increased more than 11 per cent. This had already been reflected in generally lower land values throughout Detroit's premier shopping and business section.

In making its recommendations, the Detroit Traffic Survey went well beyond the function of merely counting and analyzing traffic movements. It took into consideration both the general physical plan and organization of the city and the social and economic implications of real estate conditions in and about the central business district. New crosstown arteries were proposed to relieve the peak pressures in the central area caused by the daily movement of industrial workers. It was further proposed that the low-value areas surrounding the business district be utilized for terminal facilities.

The economic value of traffic engineering in dealing with such situations cannot be questioned. Land values are largely dependent upon accessibility. Accessibility is dependent, to a considerable degree, on the travel habits of people as well as on the physical layout and structure of streets and transportation facilities. Much has been said about street capacity for the movement of automobiles. The capacity of a street, however, is not the true measure of the potential shopping and travel habits of the people. No area is likely to attract its full quota of day-time population under conditions which exist when traffic lanes approach capacity values.

Traffic engineering is proving equally useful to the highway administrator and to the traffic enforcement officer. Cooperative assistance to these officials is not always confined to the solution of problems of traffic. It may be, and often is, applied to problems of highway finance and to details of police procedure.

The following questions confront the highway administrator, and the traffic engineer is helping him to answer them:

- What extensions to, or abandonments from, the trunk-line system are needed?
- Which highways should be supported by motor-vehicle revenues and which cannot?
- Which of the trunk lines are inadequate for existing traffic?
- What type of construction on a new highway is dictated by the character of its use?
- What will its life expectancy and maintenance costs be?
- What is the maintenance burden being accumulated on the state system, and how great can it become and still leave sufficient revenues for necessary construction?

Clearly, the answers to these questions have real economic value.

Traffic engineering is vital in the operations of the traffic division of the police department. In traffic safety and in the orderly movement of both pedestrian and vehicular traffic, intelligent enforcement methods are as important as engineering.

Enforcement agencies complain of inadequate personnel. Traffic engineering, through selective enforcement, points the way for the more efficient use of officers. The economic value of traffic engineering to enforcement agencies consists in providing the assignment officer with factual data upon which he can concentrate attention where it will do the most good; at the hours, places, and on the violations producing the majority of accidents.

Any program or plan which promotes efficiency has economic value. Just as efficiency in enforcement is obtained by selective methods, so is efficiency in educational efforts obtained through traffic studies which permit selective application. The usual safety education program for adults consists of appeals through newspapers, magazines, the radio, billboards, and safety meetings. Recent studies in Detroit, Toledo, and Louisville indicate that the residences of drivers who are most accident participating are in areas of low economic status and low responsibility. People living in these high-ratio areas are seldom interested in or capable of absorbing the usual type of safety education. The traffic engineer determines the high-ratio areas and suggests a selective approach through media which they will heed and respect.

Planning for traffic should be an important function of government and should be a continuous process. The changing needs of a state, region, or city, the shifts of population from one area to others, gradual alterations in land use, and many other variable factors, require that a traffic plan be kept constantly abreast of conditions.

FACTORS IN IMPROVING TRAFFIC CONDITIONS

To cope with this problem, consideration must be given to three different, but associated, methods of approach: (1) changes in physical conditions to make correct, safe, and expeditious practices the easiest; (2) changes in the individual's mental attitude by precept and instruction, to make safe practices common and instinctive; (3) changes in the individual's behavior by direct social pressure, to make safe practices compulsory. Reiteration may render tedious, but cannot diminish, the truth of the fact that traffic improvement involves the use of engineering, education, and enforcement. Of these three methods, the first—engineering—is the most effective, the most permanent, and the most costly. The second—education—is the most ethically desirable, but the most indirect and, for some individuals and classes, the least effective. The third—enforcement—probably will always be necessary to make up for the deficiencies of the other two methods and to handle the recalcitrant individuals—who, in traffic matters as in everything else, are always present.

That the economic value of traffic engineering may be realized to its fullest extent, a balanced traffic improvement program must include all of these methods of approach. It is because the traffic engineer has recognized the bearing of each of these factors on his job and has mastered the techniques which they involve, that he is entering a far wider field of usefulness. He is no longer a mere technical investigator and adviser for the regulatory and enforcement authorities. He has become an essential factor in all community planning. In our specialized economic society, in which highway transportation is the great integrating and correlating element, the function of the traffic engineer will have increasing importance. His activities, and the acceptance of the essential character of his services, will be based in the future, as they have been in the past, on the economic value of these services to the community.

Jet Deflectors for High-Dam Outlet Conduits

Designs for a Variety of Situations Developed by Model Tests at Carnegie Institute of Technology

By Harold A. Thomas, M. Am. Soc. C.E.
Professor, Department of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa.
and Wallis S. Hamilton, Jun. Am. Soc. C.E.
Principal Engineering Assistant, Hydraulic Research Laboratory, Carnegie Institute of Technology

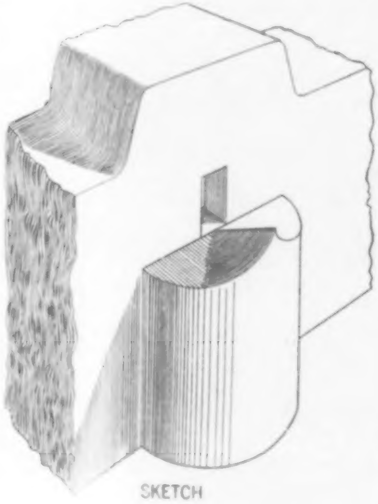
The high-velocity jets from the outlet conduits of a large dam present a serious problem of erosion control. Even if the jets fall on a concrete apron, high velocities may persist beyond the downstream limit of the paving and undercut the end sill. Satisfactory solutions of such problems can be obtained only by tests on models, and here Messrs. Thomas and Hamilton present the results of an extensive series of these tests, made for a number of War Department dams in West Virginia and Pennsylvania. As an illustration of the magnitude of the problem it may be noted that the conduits of Tygart Dam, one of the structures studied, may at full reservoir release upwards of 500,000 hp of potentially destructive energy.

Methods of minimizing the erosive potentialities of large jets have for some time been the subject of model studies in the Hydraulic Research Laboratory of the Carnegie Institute of Technology. These studies have been made for the U. S. War Department in connection with the design of dams for flood control and flow regulation in Pennsylvania and West Virginia. The structures studied include the Tygart Dam, now completed on the Tygart River near Grafton, W.Va.; the Bluestone Dam, proposed for construction on the New River near Hinton, W.Va.; and the proposed dams on Redbank and Loyalhanna creeks, tributaries of the Allegheny River in Pennsylvania.

The first of these investigations on jet control was made in 1933, during the progress of extensive model studies of various hydraulic problems pertaining to the design of the Tygart Dam. The jet issuing from each of the eight outlet conduits of this dam has a cross-section 10 ft high by 5 ft 8 in. wide, and under the maximum head of 178 ft the efflux velocity is approximately 88 ft per sec. The stilling of each jet thus requires the dissipation of energy at the rate of 37,400,000 ft-lb per sec, or 67,900 hp. It is evident that if this energy is to be prevented from expending itself in eroding deep holes in the river bed below the dam, it must be dissipated by impact against water.

In the case of the Tygart Dam the problem of providing the requisite water buffer was simplified by the decision to create a cushion pool by constructing an auxiliary dam 250 ft downstream from the toe of the main structure. Among the various laboratory methods tried for distributing the impact of the jets against the water of the cushion pool, that found most successful can be explained by a homely analogy based on the behavior of the jet from the nozzle of a common garden hose. When the gardener wishes to prevent the jet from eroding holes in a newly planted earth surface, he places his finger across the jet near the nozzle outlet in such a way as to cause the water to spread out in a thin sheet whose erosive power at any particular spot is negligible. He accomplishes this result without producing the slightest reduction in the discharge rate if he holds his finger (the "deflector") a short distance from the nozzle so that the opening is not constricted. In our studies, the word "deflector" is used to designate any object inserted into the jet to cause it to spread out into a thin sheet before striking the surface of the tailwater. No theoretical analysis was developed for the design of deflector surfaces, but by purely experimental methods, deflector shapes that would satisfactorily spread the jet were eventually determined.

The final design adopted for the deflectors at the exits of the outlet conduits of the Tygart Dam is shown in Fig. 1. Each deflector proper consists of two conical surfaces intersecting each other on the vertical plane through the axis of the conduit. These surfaces are composed of $^3/_4$-in. steel plates backed with concrete and supported on a large concrete pedestal. It is to be noted that in this structure the outlet conduits terminate in the vertical downstream face of the unusually high notched spillway apron, the design of which had been adopted previously. Figure 2 shows two views of a model representing three deflectors operating at low and medium heads. The effectiveness of the deflectors in flaring the jet into a horizontal fan-shaped sheet is clearly brought out in these figures, especially the one on the left. Figure 3 is a view of the prototype discharging



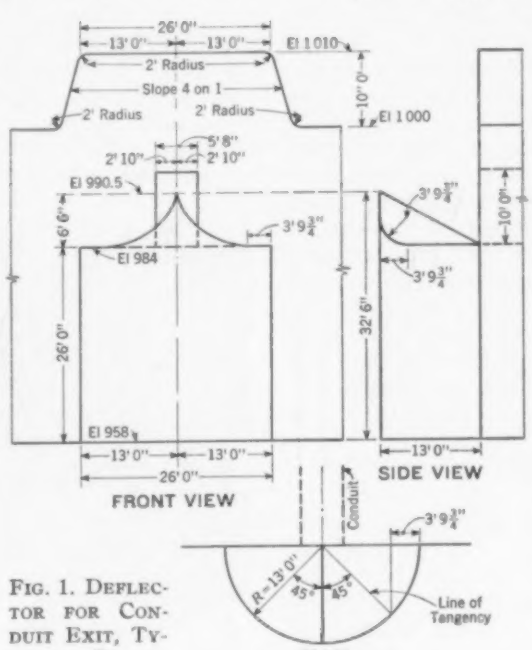


Fig. 1. Deflector for Conduit Exit, Tygart Dam

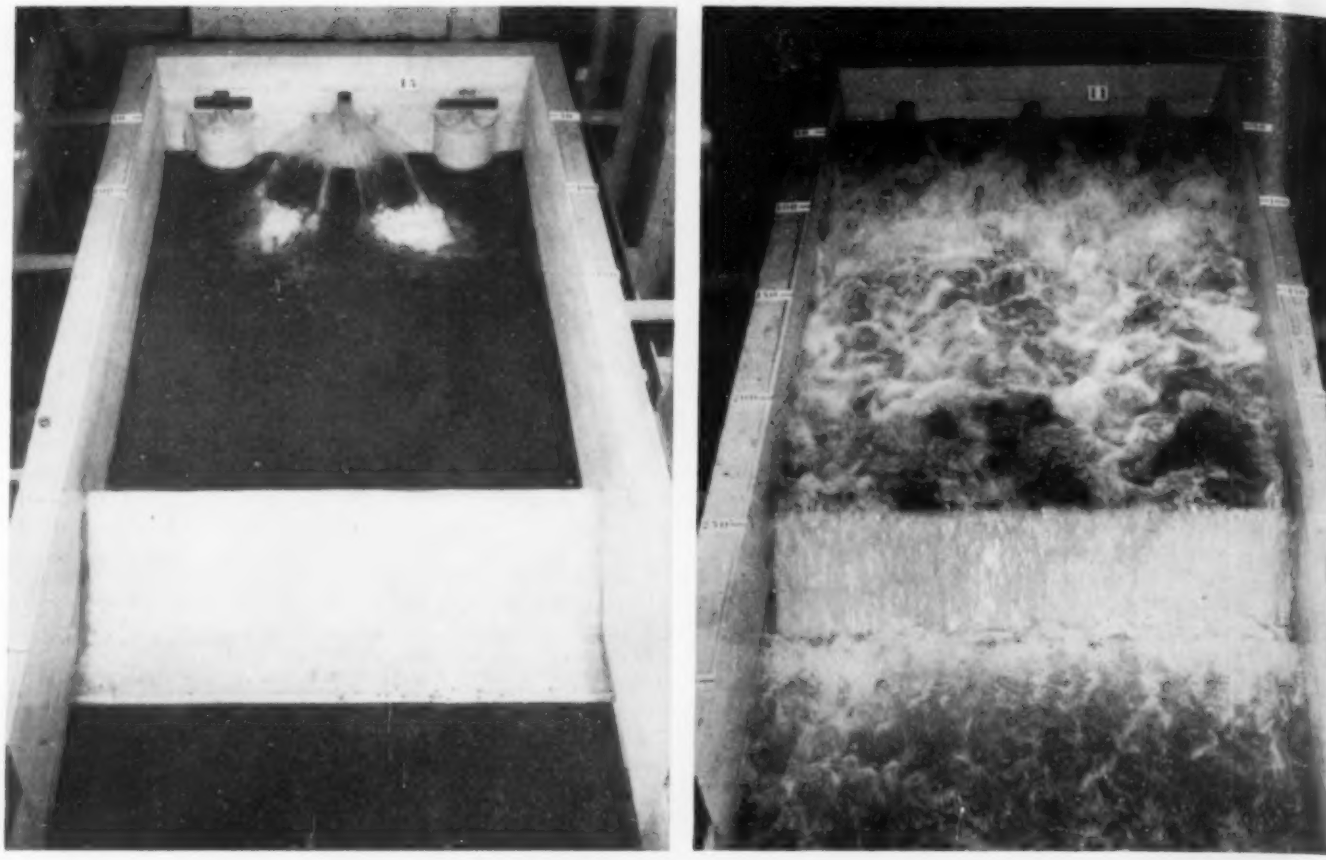


FIG. 2. TESTS ON MODEL OF THREE DEFLECTORS AT CONDUIT EXITS OF TYGART DAM
Left: One Conduit Discharging at Low Head; Stilling Pool Nearly Empty. Right: Three Conduits Discharging Under Model Equivalent of 100-Ft Head

under a head of 100 ft during the freshet of October 1937. With all but one of the conduits in operation, the tailwater is pushed away from the dam so that the deflector of conduit No. 6, still under construction, is plainly shown. The behavior of the full-sized jets is strikingly similar to that of the model jets shown in Fig. 2, and the similarity is brought out even more impressively in motion pictures showing the models and prototype in action.

During the tests on the Tygart deflectors, it was determined that the energy-dissipating action was best when the tailwater elevation in the cushion pool was near the midpoint of the conduit exits. If the conduits were submerged, turbulence and surface velocities in the cushion pool were decreased, and a corresponding increase in bottom velocities occurred. On the other hand, low tailwater permitted the trajectory of the fan-shaped sheet of water to gain too much of a downward component before meeting the cushion pool, and again rather high bottom velocities resulted where the jet dug into the tailwater. The elevation of the conduit exits with respect to the crest of the secondary dam was therefore established so as to produce 50 per cent submergence of the exits under average conditions.

BLUESTONE DEFLECTORS LIKE THOSE AT TYGART, BUT WITH "GABLES"

In 1935 extensive model studies were made at the Institute in connection with the design of the proposed Bluestone Dam. Among them were experimental investigations to develop suitable deflectors to aid in dissipating the energy of the jets from the eight rectangular outlet conduits. As in the case of the Tygart Dam, these sluices were 10 ft high and 5 ft 8 in. wide in cross-section. The deflector surfaces finally recommended were practically identical with those for the Tygart Dam. However, in the case of the Bluestone Dam, the conduit exits emerged from the bucket or sloping portion of the spillway apron, and hence the deflector projected into the path of the spillway discharge. For this reason it was necessary to elevate a portion of the sloping part of the ogee spillway surface in the form of a gable or penthouse immediately over each conduit outlet, in order to prevent the deflector from being struck by ice or logs passing over the spillway during floods.

Dimension details of the Bluestone outlet are shown in Fig. 4. Figure 5 shows how the sheet of water from the Bluestone outlet enters the cushion pool, the energy

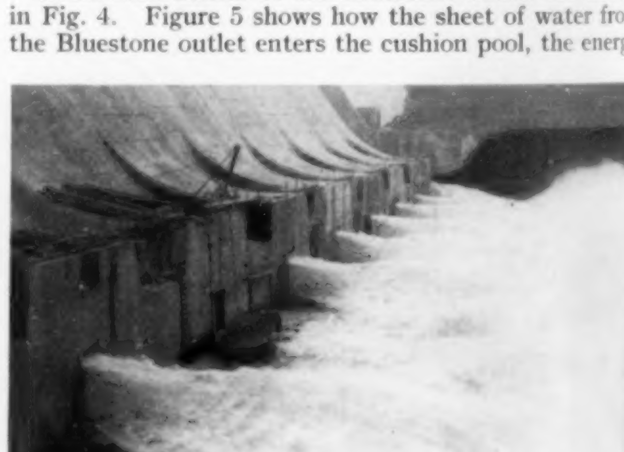


FIG. 3. TYGART DAM CONDUITS OPERATING UNDER 100-FT HEAD
This View, Taken When the Deflector for Sluice No. 6 Was Still Incomplete, Shows Clearly the Method of Construction

of the jet being dissipated near the surface so that little turbulence reaches the gravel bed of the model.

In 1938 numerous experiments were conducted on models of the proposed flood-control and river-regulation dam on Redbank Creek, among these being tests to develop the best possible deflector designs for the conduit outlets. Since only minor changes were involved, the Redbank deflector was subsequently adapted to the conduits of the Mahoning Dam—a concrete flood-control structure proposed for construction on Mahoning Creek, another tributary of the Allegheny River. The original design proposed for the Redbank Dam included five conduits 10 ft high by 4 ft 6 in. wide in cross-section, intended to operate under a maximum head of 116 ft, while the final Redbank - Mahoning conduit tested was 10 ft high by 5 ft 8 in. wide and operated under a maximum head of 142 ft when simulating the latter dam.

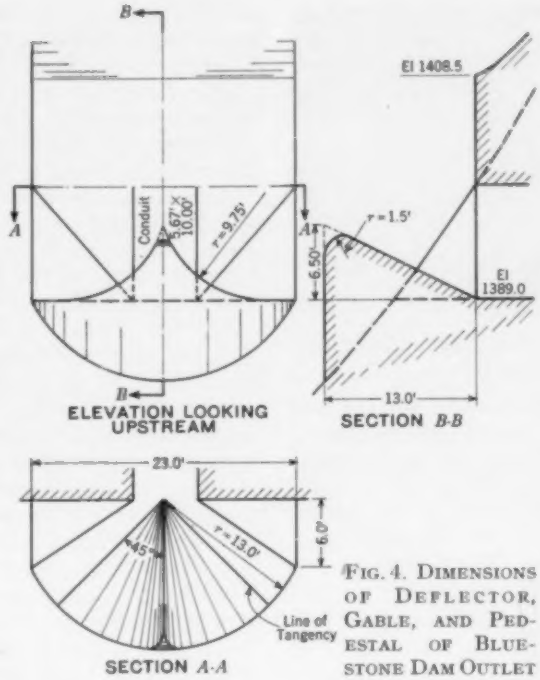


Fig. 4. Dimensions of Deflector, Gable, and Pedestal of Bluestone Dam Outlet

As adequate but variable tailwater depths were present in Redbank Creek at the dam site, no cushion pool dam was incorporated in the project design. Since the Tygart and Bluestone deflectors had been specifically designed for cases where variation in tailwater elevation at the conduit exits was controlled within close limits by cushion pool dams, it was considered necessary to make additional tests to determine the behavior of deflectors of this type under a wide range of tailwater elevations, and also to investigate the possibility that some totally different exit design might give better results under the conditions encountered in Redbank Creek.

With regard to the latter point, the conclusion determined from numerous tests was negative. At all tailwater depths, the best deflectors were found to be those which resembled the original ones at Tygart in possessing the ability to spread the jet out into a thin horizontal fan-shaped sheet. While the optimum energy-dissipating effect of such deflectors occurred with the tailwater stage about at the center of the conduit outlets, nevertheless their action was quite satisfactory for the range of tailwater occurring at Redbank Dam, and was definitely better than could be obtained with exits or deflectors of any of the other types tested. The absence of scour in the Redbank model was partly due to protection of the river bed by a long concrete apron, which had been previously designed on the basis of spillway requirements. Under normal tailwater conditions all water of each deflected jet met the tailwater surface at some point upstream

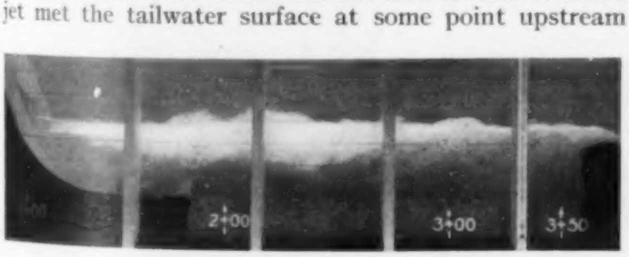


Fig. 5. Sectional Model of Bluestone Spillway, with One Conduit Operating at Maximum Head (126 Ft in Prototype)

Note the High Surface Velocities and the Absence of Scour

from the end sill of the apron, where local high velocities could cause no damage. However, the securing of approximately uniform lateral distribution of flow over the end sill, as provided by the deflectors here described, was found necessary in order to prevent undercutting at the downstream limit of the concrete. In the model severe cutting occurred at this place when the deflectors were removed from the conduit exits.

The Redbank Creek Dam resembles the Bluestone Dam in that the conduit exits emerge from the bucket or sloping portion of the spillway. The first deflectors designed for the Redbank Creek Dam were therefore modeled closely after those for the Bluestone Dam, as shown in Fig. 6. However, in the interest of economy, the designing engineers of the Redbank Creek Dam decided to eliminate the gables over the conduit exits. The members of the staff of the Hydraulic Research Laboratory were therefore requested to develop a deflector design that would involve no projection of concrete beyond the ogee face of the spillway. The models on the left and right in Fig. 7 show two preliminary designs that were studied in the effort to achieve this result, while Fig. 8 shows a transparent pyralin model built in accordance with the design finally recommended.

Fig. 6. Preliminary Design of Conduit Outlets for Redbank Creek Dam, Under Test in 1:24 Model

In the first attempts to move the deflector upstream until no part of it should project beyond the surface of the ogee spillway, the shape of the upper nappe of the jet from the Bluestone-type deflector shown in Fig. 6 was measured and then reproduced in plastic material to form the roof and walls at the exit of a new model conduit. While the shape of the resulting fan-shaped jet duplicated that of the original jet quite accurately, the design had the disadvantage of requiring intricate form work to produce concrete surfaces of complex double curvature. Subsequent tests on an extensive scale developed the fact that excellent performance could be obtained from deflector and outlet combinations in which the design was so simplified as to involve plane surfaces connected by either warped or singly curved transitions. This feature is possessed by the design finally recommended, the deflector surface proper being especially simplified to involve only two triangular planes instead of the two conical surfaces used in previous designs. The deflector proper thus became a simple tetrahedron.

It was not found feasible to develop the design of the Redbank and Mahoning conduit exits and deflectors by purely analytical methods, because the boundary condi-

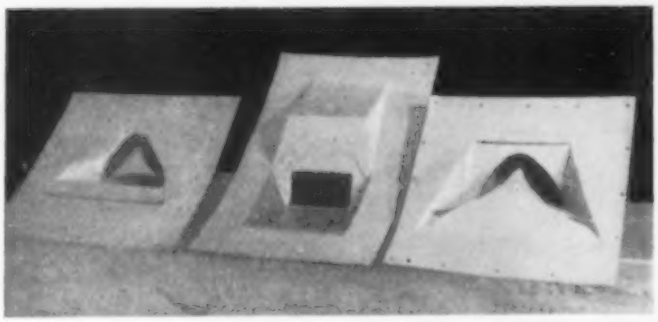


FIG. 7. OTHER PRELIMINARY DESIGNS FOR REDBANK CREEK DAM CONDUIT EXITS AND DEFLECTORS

tions pertaining to the curved surfaces of the solid materials and of the free jet were too intricate for practical use in analytical expressions. However, in the preliminary designs an effort was made to apply the principle of maintaining equality of cross-sectional areas normal to the estimated direction of flow. The passages thus designed were found to require more or less subsequent revision by experimental methods.

A careful investigation was made of the pressures existing at various points along the solid surfaces, numerous piezometer holes being drilled through the model walls for this purpose. Of course the scale of atmospheric pressures was greatly exaggerated in the models, and it was recognized that for this reason the prototype jets might tend to leave the solid surfaces at certain places where the model jets would maintain contact. While this effect could have been investigated by enclosing the models in a vacuum tank, that expensive step was not actually taken, but was replaced by a careful study of piezometer pressures at numerous points and by the provision of a reasonable margin of safety in the design at places where the pressures tended to be low.

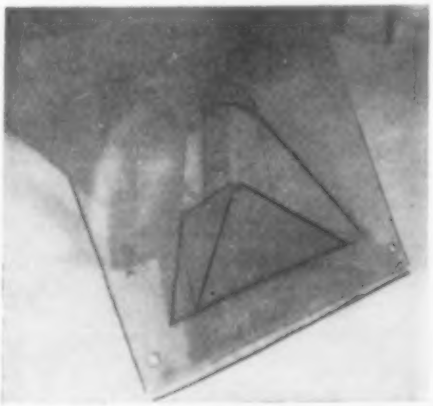


FIG. 8. TRANSPARENT PYRALIN MODEL (SCALE 1:24) OF THE TYPE OF EXIT AND DEFLECTOR RECOMMENDED FOR REDBANK AND MAHONING DAMS

CONDUIT PRESSURES AFFECTED BY EXIT DESIGN

It was found that pressures throughout the conduit length could be materially modified by making comparatively small changes in the dimensions of the exit passages. Too great a constriction at the exit had an effect similar to that of a nozzle in building up the conduit pressures, while too great a flare had the opposite effect, or that of a diverging tube. The former effect was considered undesirable because of the consequent reduction in conduit discharge capacity, and the latter because of the increased likelihood of cavitation and the possibility of the formation of an unsymmetrical jet due to failure of the water to cling to both side-wall curves.

Figure 9 is an overhead photograph of the jet issuing from the Redbank deflector model. The tailwater elevation corresponds to that for the discharge from a single conduit, being only 2 ft above the outlet floor, but the energy dissipation is satisfactory. Figure 10 shows a general model of the Redbank Creek Dam with the extreme left-hand conduit (looking downstream) in operation. As shown by the confetti streaks, a large eddy exists below the dam in cases where the conduits in operation are unsymmetrically located with respect to the center line of the channel or cushion pool. Similar eddies were observed in the Tygart cushion pool during experimental studies conducted on the Tygart general model. It was demonstrated that the installation of deflectors at the Tygart conduit exits greatly diminished the severity of these eddies, the reduction in bottom velocities being especially pronounced.

The tests here described for the Tygart and Redbank dams, together with those now in progress on deflectors for the Loyalhanna Dam, were conducted for the Pittsburgh Office of the Corps of Engineers, U. S. Army, by the Hydraulic Research Laboratory at the Carnegie Institute of Technology, which laboratory is under the supervision of the writers. The work on the development of the deflectors was carried on in close cooperation with members of the Dam Design and Projects Sections of the Pittsburgh and Huntington offices of the Corps of Engineers. The evolution of the outlet designs as finally adopted was due to the combined efforts of numerous persons. Lt. Col. W. E. R. Covell, M. Am. Soc. C.E., is the district engineer at Pittsburgh; E. M. Wellons is the principal engineer responsible for designs; E. P. Schuleen, Assoc. M. Am. Soc. C.E., engineer, has general charge of matters pertaining to model tests; and W. J. Hopkins, Jun. Am. Soc. C.E., assistant engineer, is the resident engineer at the laboratory. The Bluestone tests were conducted for the Huntington Office, for which Lt. Col. John F. Conklin was district engineer during the period of the model studies.

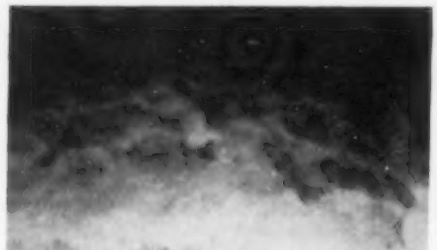


FIG. 9. REDBANK MODEL IN OPERATION AT LOW TAILWATER

The Condition Simulated Here Is Discharge at a Head of 78 Ft, with Tailwater 2 Ft Above Conduit Floor Level

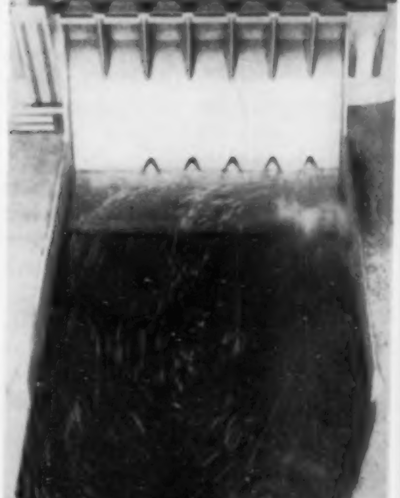


FIG. 10. REDBANK "GENERAL" MODEL (SCALE 1:72) WITH ONE CONDUIT IN OPERATION

The Manufacture of Iron and Steel

Part II—Finished Products

By F. H. Frankland

Member American Society of Civil Engineers

Chief Engineer, American Institute of Steel Construction, Inc., New York, N.Y.

The past ten or fifteen years have seen the development of many new iron and steel products and the improvement of many old ones. New practices in manufacturing have been evolved—outstanding among them the continuous sheet mill, "greatest single invention to date in the steel industry." With the present article, discussing finished products and methods of production, Mr. Frankland concludes his concise word picture of steel manufacture. The first installment, describing the production and purification of pig iron, appeared in the April issue of "Civil Engineering."

In order to understand the practices common in the manufacture of steel and iron, it is necessary to be familiar with the physical characteristics required in the finished product. Among these properties, yield strength and ultimate strength are two of the most important. In the design of a compression member (within the lower ranges of l/r, where elastic bending does not control), yield strength is all important because it is the determining factor with respect to possible ultimate failure. On the other hand, in applications where failure might be due to repeated stresses induced either directly or by vibration, we are less interested in yield strength than in the fatigue or endurance limit, which is a direct function of the relation between yield point and ultimate strength.

The yield point and ultimate strength of steel depend not only upon the chemical composition of the material, but also upon the amount of work done on the material during rolling, and the temperature at which the work is finished. In general the yield point increases as the rolling temperature is lowered. Again, if steel is loaded to produce stresses beyond its yield point (thus producing permanent set), the material will possess a higher yield point after the original stresses are removed. It is this latter principle that is used in the manufacture of cold-rolled and cold-drawn steel, such as the wire used for suspension bridge cables.

It is important to emphasize here how greatly the actual internal properties of steel affect the strength of a welded structure. We should not base our studies in such instances on the stress-strain diagram of standard tension test specimens, and we should realize that the difficulties of welding structural steel are not of a theoretical so much as of a material and mechanical nature. These difficulties begin where the basic work laid down by the theory of elasticity ends, and the all-important problems related to actual behavior of the material under the various combinations of circumstances must be considered.

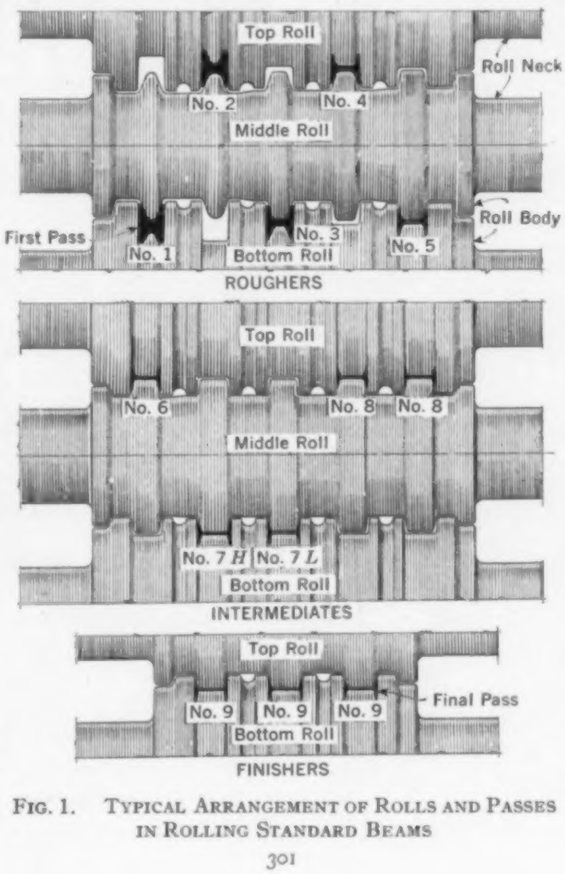


Fig. 1. Typical Arrangement of Rolls and Passes in Rolling Standard Beams

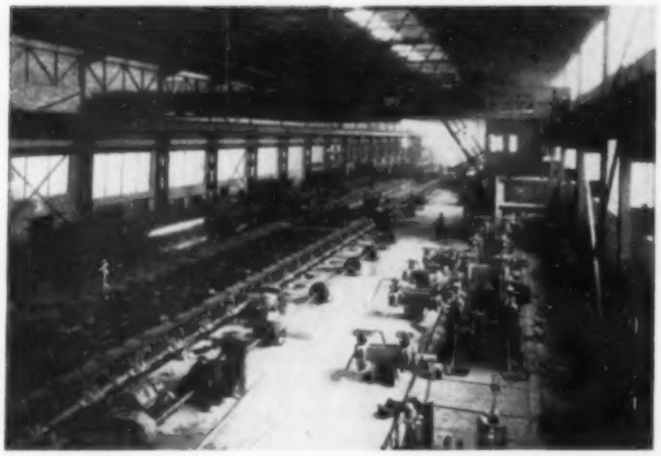
The effect of carbon upon the yield point and ultimate strength of steel is to increase the values of both. At the same time, however, it makes the steel harder and less ductile, and for that reason in ordinary structural grades it is usually held below 0.30 per cent. Manganese also has the effect of increasing the yield point and tensile strength, but not to the same extent as carbon. If present in large quantities it produces a material difficult to manufacture. One of the important purposes of adding manganese is to neutralize the evil effects of sulfur. Sulfur up to 0.1 per cent has little or no influence on the ductility or strength of steel at ordinary temperatures; it does, however, produce "red shortness"—that is, it makes the steel difficult to work at a red heat.

Phosphorus is responsible for steel being "cold short," or brittle at ordinary temperatures, but it is doubtful whether quantities less than 0.1 per cent have any marked influence except upon the weldability of the steel. It does increase the hardness and tensile strength of steel, but reduces the ductility.

Silicon is considered beneficial up to about 0.75 per cent, because up to this point it increases the yield point and ultimate strength without reducing ductility. However, it probably has the effect of reducing the endurance limit.

The use of copper in amounts of 0.20 to 0.40 per cent, to produce increased resistance to atmospheric corrosion in steel, has been economically established for some years. The value of a larger percentage of copper up to a saturated solution, as a means of increasing strength, has but recently become widely recognized, research having demonstrated that copper is a strength-giving alloy for steel. No noteworthy increase in strength is produced until the copper content reaches 0.50 per cent, but above that the increase is marked, as is demonstrated in several of the newer high-strength low-alloy structural steels.

It is difficult for one not entirely familiar with the operations in steel manufacture to appreciate the innumerable details that must be taken care



Bethlehem Steel Co.

ELECTRICALLY OPERATED STRUCTURAL STEEL MILL AT LACKAWANNA, N.Y.

of and the conditions that must be fulfilled in every step of the process. When, however, it is understood that between the blast furnace producing the pig iron, and the rolling mill producing the finished steel, there is a loss of production amounting, in many cases, to from 25 to 30 per cent, the uninitiated will realize to some extent what is involved when steel of a special analysis is required. For every ton of finished rolled steel that is shipped from the mill, approximately six tons of ore, fuel, fluxes, refractories, lubricants, and so forth, must be shipped into the mill.

The following table of iron and steel products produced in the United States in 1938 indicates the general distribution today:

STEEL PRODUCTS	GROSS TONS
Ingots, blooms, billets, slabs, sheets, bars, etc	1,708,067
Heavy structural shapes	1,387,204
Steel piling	114,764
Plates—sheared and universal	1,457,588
Skelp	394,977
Rails and fittings	808,829
Bars, hoops, and bands	3,019,526
Tool steel bars (rolled and forged)	16,541
Pipe and tube	2,152,755
Wire and wire products	2,001,831
Black and tin plate	1,707,515
Sheets	4,507,415
Strip—hot and cold rolled	1,317,083
Wheels, axles, and track spikes	150,904
All other	48,316
Total steel products	20,793,315
IRON PRODUCTS	
Bars	16,951
Pipe and tubes	26,394
All other	6,797
Total iron produc's	50,142

It will be noted that an important part of steel-mill production is in the form of blooms, billets, slabs, and sheet bars, as well as ingots worked directly into finished forgings in those cases where blooms would be inadequate in size. Blooms, billets, and slabs are hot-rolled from ingots to approximately the required cross-section; blooms and billets are square or rectangular, and slabs have the shape of flat rectangles in section. Sheet bars are flat sections with rounded edges, rolled to weights of 7 to 54 lb per lin ft and widths of 8 to 16 in. They are used for re-rolling into sheets and black plate, and are commonly the product used by the tin plate mills. Ingots vary considerably in shape and size, but are usually square or rectangular in cross-section. Those from which slabs are to be rolled generally have a width more than twice the thickness, so that the amount of reduction in the rolling process can be limited to that needed to produce the required physical characteristics.

Steel for structural purposes includes flanged sections, plates, and rivets, as well as a new but so far limited use of sheet steel such as is produced on the continuous sheet mill. The flanged sections known as structural shapes or sections are of two types—American Standard and Wide Flange. The American Standard sections were established in 1896 and cover series of I-beams, channels, and angles; Wide Flange sections were developed in the first decade of the century and include I- and H-shapes. Channels, angles, tees, and zees, 3 in. or less in maximum cross-sectional dimension, are called bar-size sections, and are used principally for other than purely structural purposes.

To produce flanged structural sections, blooms, billets, or slabs are passed through a series of grooved rolls. A typical arrangement of rolls and passes for standard beams is shown in Fig. 1. The approximate dimensions and shape of the sections are obtained on the roughing rolls, and the finished product results from the passes of the intermediate and finishing rolls. Typical stages in the rolling of angles and channels are shown in Fig. 2. The ingot is heated to the proper temperature and rolled into blooms or billets, which are then reheated for working by the structural mill.

In order to roll wide-flange sections, a special mill (Fig. 3) is required, as steel at rolling temperatures does not flow readily and therefore must be worked to form wide flanges. The wide-flange mill differs from the standard structural mill in that vertical rolls are substituted for grooves. A study of the schematic arrangement illustrated in Fig. 3 shows how the preliminary work on the section sides and edges is performed alternately in the roughing and intermediate stands, while the finishing passes work the section faces down to exact dimensions.

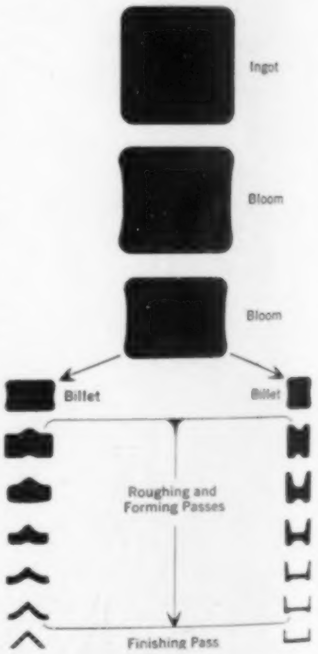


FIG. 2. TYPICAL STAGES IN ROLLING OF ANGLES AND CHANNELS

It is impracticable to achieve faultless rolling-mill practice, and the conditions are such that certain obligations are imposed upon the purchaser as well as the manufacturer. Rolling mills usually make free replacement of defective material at the original point of delivery even though it may have passed inspection at the mill by the purchaser's representative; they do not, however, accept liability for charges other than freight, or for fabricating expenses incurred upon material rejected after delivery.

Structural-quality steel is governed by the requirements of the following standard specifications of the American Society for Testing Materials:

- A-7. Steel for bridges
- A-8. Structural nickel steel
- A-9. Steel for buildings
- A-10. Mild steel plates
- A-94. Structural silicon steel
- A-113. Structural steel for locomotives and cars
- A-131. Structural steel for ships
- A-141. Structural rivet steel
- A-195. High-strength structural rivet steel

Of these specifications, A-10 and A-141 have been approved as "American Standard" by the American Standards Association.

The continuous mill for producing steel sheets and strips, which has been developed during the past 15 years, has been called the greatest single invention to date in the steel industry. It is epoch-making in its economic effects. Without the high-quality, low-cost steel sheets produced by it we would not be able to manufacture modern automobiles, modern electric refrigerators, modern gas ranges, or literally hundreds of other products in everyday use. The continuous sheet mill is directly responsible for the tremendous growth in the use of sheet steel during the past ten years, such use having more than doubled in this period, until 7,000,000 tons were produced by this process in 1938.

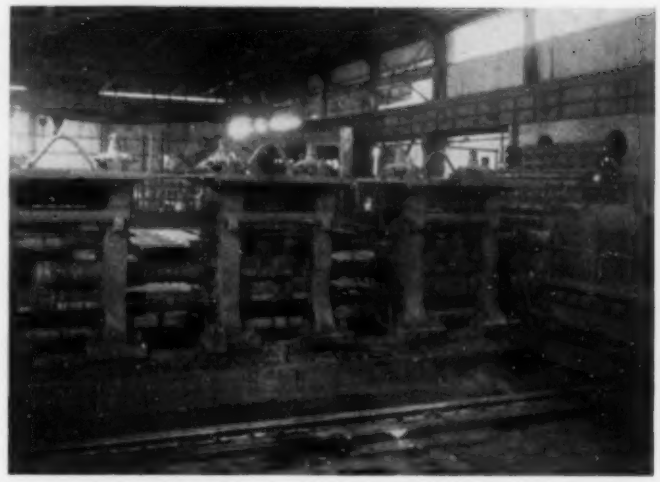
In 1923, when the American Rolling Mill Company put the first continuous mill into operation at Ashland, Ky., the usual prophecies were heard regarding the unemployment that would result from technological advance. However, the number of men employed in the production of steel sheets has increased 28 per cent during the past ten years. Add to this the great number of men working in the industries made possible by the use of sheet steel produced on continuous mills, and you have what is undoubtedly the greatest single factor in increasing employment in history.

Production of Steel Sheets

The operation of the continuous sheet mill is truly continuous from the ingot to the finished product. In a typical mill the ingot goes from the soaking pit to the blooming mill, the product of which is the slab. The slab is then cut to requisite length on the slab shear to suit the mill production schedule, and passes to a double-ended furnace where it is reheated to proper rolling temperature. From this point the sheared slab is successively reduced in thickness in the roughing stands to bar plate size—$1/8$ to $1/10$ the original thickness and 8 to 10 times the original length. The bar plate goes then to the finishing stands of the hot strip mill, where it is reduced to "wide strip," which is coiled for sorting in various thicknesses.

The hot coiled strip next passes to the uncoiler, which feeds it to a shear, after which a stitching machine joins the coils together into a continuous ribbon. The latter is then pulled, by pinch rolls, through the pickling tanks, where a sulfuric acid solution removes the scale and cleans the surface. From the pinch rolls the wide strip goes through a second shear, where the stitched joints between the coils are sheared out. It then passes through side shears that trim the edges, and is coiled for storage.

These coils next pass, through an uncoiling machine, to the cold reduction mill, where the cold wide strip is reduced in thickness by from 35 to 75 per cent and again coiled. Another uncoiling machine feeds this material through a flying shear, where it is cut accurately to strip sheets of the required length. The latter then pass through an annealing furnace, for proper heat treatment to develop the physical properties required for the various uses to which they are to be put. Next they are rolled in a temper mill to produce the required temper, surface, and flatness, and thence they pass through a roller leveler to the inspection table. After inspection, the sheets are ready for shipment.



Colorado Fuel and Iron Co.

Roll Stands in a 25-In. Structural Mill
A Traveling Tilting Table Operates in the Pit in the Foreground

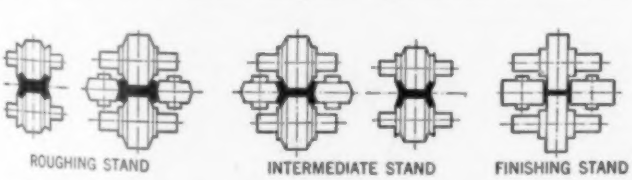


Fig. 3. Schematic Arrangement of Mill Rolling Wide-Flange Beams
In the Roughing and Intermediate Stands, Alternate Passes Work on Whole Section Except on Flange Edges, and on Flange Edges Only

Standard Dimensions for Steel Plate

Plates are flat, hot-rolled, finished steel products made to the following dimensions as established by the American Iron and Steel Institute and the Association of American Steel Manufacturers:

$3/16$ in. or thicker, over 48 in. wide
$1/4$ in. or thicker, over 6 in. wide
7.65 lb per sq ft or heavier, over 48 in. wide
10.2 lb per sq ft or heavier, over 6 in. wide

(Unfinished steel products within this size range, not classed as plates, include slabs, sheet bars, and skelp.)

Plates may be hot rolled either from slabs or direct from ingots. They are usually classified as "sheared" and "universal"; the former are rolled between horizontal rolls only and trimmed on all edges, while the latter are rolled between horizontal and vertical parallel rolls and trimmed on the ends only. Chemical and physical requirements for plates and tolerances as to dimensions are covered by standard specifications for plates, such as those for structural quality already quoted, and in addition the following:

A.S.T.M. A-70. Flange (boiler) quality and firebox (ordinary) quality
A.A.R. M-115. Locomotive firebox quality

Marine-boiler quality plates are covered by the specification of the Bureau of Marine Inspection and Navigation, U. S. Department of Commerce.

The specifications of the American Society for Testing Materials covering steel plates include tables of permissible variations of rectangular plates ordered to weight, permissible overweights of rectangular plates ordered to thickness, and so forth. The Association of American Steel Manufacturers has also adopted tables of permissible variations in thickness, length, width, flatness, and camber for plates over 2 in. thick. The requirements of these two standardization bodies do not conflict but supplement each other.

For the purposes of this article, steel tubular products may be classified as pipe, tubes, tubing, and casing. Pipe and tubes are produced (1) by the seamless process, (2) by the welding process, and (3) by shop fabrication.

The seamless process involves either "hot piercing" or "cupping." In the hot piercing method the ingot, bloom, billet, or round bar is passed between rolls set at a slight angle to one another, so that a rotating as well as a forward motion is imparted to the metal. A tapered piercing mandrel, pointed toward the entering side of the rolls and extending just beyond the point of minimum clearance, penetrates the steel as it moves forward. Lengths up to 40 ft can be produced in this manner. Additional rolling is of course required to produce finished tubes.

Weirton Steel Co

COILING FROM THE COLD STRIP MILL
One Coil of Such Sheet Steel Forms a Strip Nearly a Mile Long

For the cupping process, a plate of the proper thickness is cut into circular shape, heated to the required temperature, and "cupped" by a plunger which forces it through a die. The "cup" is sheared off and the remaining tube is hot drawn until the finished size is attained. When small sizes are required with great accuracy in dimensions, tubular products are generally finished by cold drawing.

Large pipes and tubes are frequently made from plate material by butt-welding or lap-welding. In butt-welding the skelp is heated throughout its length to welding temperature and then drawn through a die or through welding rolls into tubular form, the edges being pressed together to make the weld. In lap-welding the skelp is heated as for butt-welding but is bent into tubular form with the edges overlapping. Reheating to welding temperature is then necessary before passage through the welding rolls.

Large pipe is frequently welded by fusion-welding or electric-resistance-welding, or by forge-welding or hammer-welding. Some large pipe is fabricated in the shop by riveting or by the use of special patented edge locks.

CLASSIFICATION OF TUBULAR PRODUCTS

The usual classifications of tubular products are (1) standard pipe, (2) pipe classified by grades and uses, (3) oil country tubular goods, (4) pressure tubes, and (5) mechanical tubing. When pipe or tubes are to be specified by dimensions, care should be taken to consult the manufacturers' tables and the applicable standard specifications, as some differences exist between actual and nominal dimensions. Tubular products are variously specified by their inside or outside diameters.

The quality, dimensions, and uses of standard pipe and tubes are covered by A.S.T.M. Specifications A-53, A-120, A-135, and A-139. The A-120 specification covers butt-welded, lap-welded, electric-welded, and seamless pipe for ordinary purposes. The other specifications cover pipe for special purposes, as follows: A-53, butt-welded, lap-welded, and seamless pipe; A-135, electric-resistance-welded pipe; and A-139, electric-fusion-welded pipe.

Casting is one of the oldest methods for making metal parts, and is still used extensively, though there is a steady increase in the use of forgings and welded assemblies in place of castings. The casting process involves the use of a pattern of wood or metal, similar in shape to the desired finished piece, but slightly larger in dimensions to allow for shrinkage of the metal when it freezes. The pattern is first bedded in molding sand, and then removed, after which the metal is poured into the mold through gate openings. Following the solidification and cooling of the metal, the mold is broken away and the casting removed.

The past few years have seen the substitution of welded plate and structural section assemblies for many products that were formerly made of iron or steel castings. By such substitution a more uniform quality of material can be secured, as welded assemblies can be annealed to remove residual welding stresses, and the blowholes common in castings can be definitely avoided. The cost per pound of such welded assemblies is invariably considerably less than that for castings for the same use.

The American Society for Testing Materials has issued the following specifications covering the various uses to which steel castings are put:

STANDARD SPECIFICATIONS

A 87-36.	Carbon-steel and alloy-steel castings for railroads
A 95-36	(Approved as "American Standard" by the American Standards Association.) Carbon-steel castings for valves, flanges, and fittings for high-temperature service
A 128-33.	Austenitic manganese-steel castings
A 148-36.	Alloy-steel castings for structural purposes
A 157-36.	Alloy-steel castings for valves, flanges, and fittings for service at temperatures from 750 to 1,100 F

TENTATIVE SPECIFICATIONS

A 27-36T.	Carbon-steel castings for miscellaneous industrial uses
A 168-35T.	12 per cent chromium steel castings
A 169-35T.	19 per cent chromium steel castings
A 170-35T.	28 per cent chromium steel castings
A 198-36T.	20 per cent chromium, 9 per cent nickel alloy steel castings
A 171-35T.	24 per cent chromium, 12 per cent nickel alloy steel castings
A 172-35T.	25 per cent chromium, 20 per cent nickel alloy steel castings
A 173-35T.	28 per cent chromium, 9 per cent nickel alloy steel castings
A 174-35T.	20 per cent nickel, 9 per cent chromium alloy steel castings
A 175-35T.	35 per cent nickel, 15 per cent chromium alloy steel castings

It will be noted that in this list of specifications there are 11 covering alloy-steel castings. Castings made from such materials are not susceptible to replacement economically by welded assemblies.

Carnegie-Illinois Steel Corp.

A 42-IN. FIVE-STAND COLD REDUCTION MILL IN THE IRVIN WORKS, NEAR CLAIRTON, PA.

Genesee River Flood Control at Rochester, N.Y.

By Edwin A. Fisher, Hon. M. Am. Soc. C.E.
Consulting Engineer (Retired), Rochester, N.Y.
and Perfecto A. Covas, Assoc. M. Am. Soc. C.E.
Assistant Engineer, Department of Public Works, City of Rochester, Rochester, N.Y.

By Supreme Court decision, a river is an amenity; it may also be a liability. The Genesee made Rochester, N.Y., a thriving manufacturing center and now threatens it with floods. The nub of the problem is at the business center, where encroachments have restricted the waterway. To provide for an anticipated flood of 90,000 cu ft per sec it is necessary to make the channel wider, deeper, and smoother; to enlarge bridge openings; and to raise protective walls. This description of a $4,000,000 project has been condensed from a much longer paper which was delivered before the Rochester Meeting of the Society.

The Genesee River rises in Pennsylvania about 15 miles south of the New York State line and flows northerly across the state of New York through the heart of the city of Rochester, and into Lake Ontario at the northerly city limits. The total length of the watershed is about 118 miles, and the average width is 22 miles. The drainage area above Rochester is about 2,435 sq miles.

At Rochester there are three falls with a total drop of 245 ft. According to Professor Emeritus Herman Leroy Fairchild, of the University of Rochester, "The Genesee River is the cause, explanation, or apology for the city of Rochester. No river, no cataract, no power, no city." The place was incorporated as a village in 1817 with a population of about 1,000, and as a city in 1834, with a population of about 12,000. Its present population is about 340,000.

The old Erie Canal, extending from Buffalo to the Hudson and opened in 1825, was a potent factor in the development of the city. Its successor, three miles south, is the Barge Canal, opened to navigation in May 1918. The Erie Canal lands through the city were purchased by Rochester in 1922 and are now occupied by the city-owned Rapid Transit Railroad. This subway crosses the river at Broad Street (Fig. 1) as the canal did before it.

After a long study of flood conditions in the Genesee River, the late John R. Freeman, Past-President and Honorary Member Am. Soc. C.E., concluded that 90,000 cu ft per sec is a reasonable estimate of a future flood through the city under present conditions in the watershed. Separate factors caused the floods of March 1916 and March 1913. A combination of these same factors, but with less intensity, accounted for the flood of March 1865. A recurrence of this earlier flood, with a flow equal to the combined effect of the factors that caused the later ones, may be expected in the future to give a possible peak flow in excess of 90,000 cu ft per sec, as anticipated by Mr. Freeman.

It is estimated that a flood of 90,000 cu ft per sec would require a three-day storm totaling $7^1/_4$ in. of precipitation over the drainage area above the city. This assumes existing conditions of natural storage and a ground saturation such as to yield a 90 per cent runoff. The estimated natural storage on the watershed at the time of peak flow would be 14,000,000,000 cu ft distributed over a natural storage area (valley and lakes) of about 100 sq miles. About 60 sq miles would be included in the valley of the Genesee River and Canaseraga Creek between the Barge Canal at Rochester and Dansville (45 miles long), exclusive of about 20 sq miles in the tributary area. The estimated average depth of extra storage in the 60 sq miles is about $6^1/_3$ ft, and on the remaining 40 sq miles, approximately 3 ft.

Various investigations were made by legislative commissions, city council committees, and private organiza-

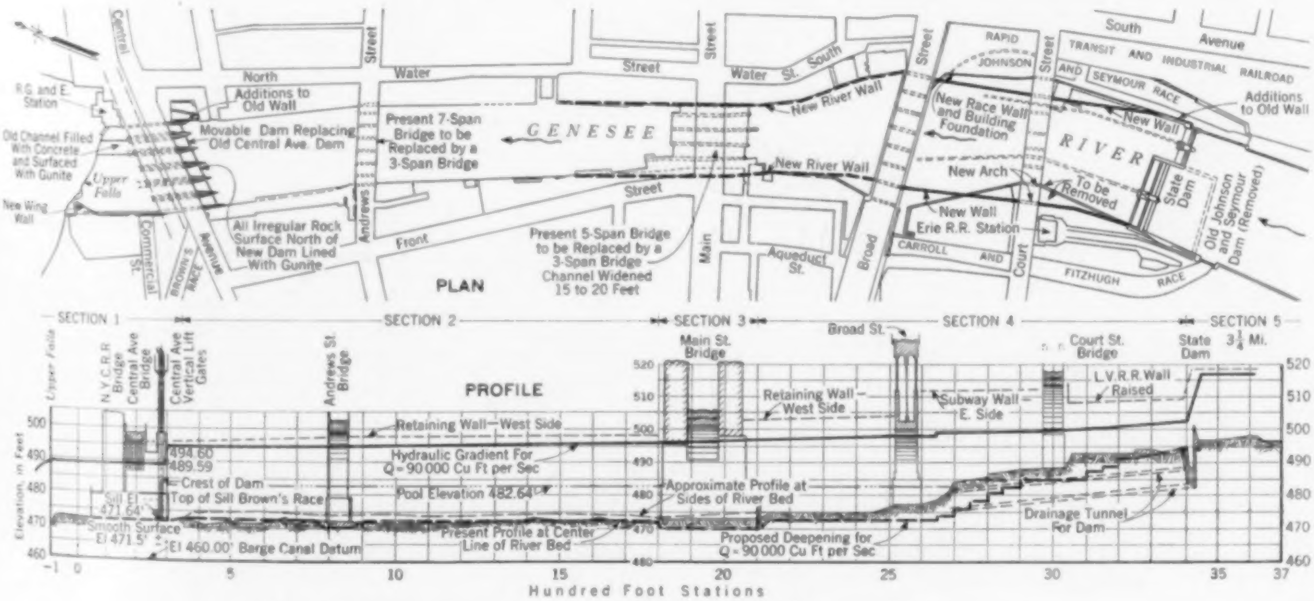


Fig. 1. Plan and Profile of Genesee River Through Rochester
Showing Proposed Improvements to Enlarge Capacity to 90,000 Cu Ft per Sec

tions, as to the best method of alleviating the flood situation in Rochester. These may be summarized briefly under the headings of (1) water storage in the valley above Rochester, and (2) channel improvement in the city of Rochester. A comprehensive plan for a maximum capacity of 60,000 cu ft per sec was recommended by the Flood Committee of 1904.

RIVER CHANNEL LOOKING NORTH (DOWNSTREAM) TO UPPER FALLS, SHOWING CONSTRICTION AT MAIN STREET BRIDGE

River Crossings Shown Are (from Bottom) Court Street; Broad Street (Subway on Lower Deck); Main Street (with Business Buildings Over River); Andrews Street; and Central Avenue (New York Central Railroad Parallel and Adjacent). Beyond This Point the River Is in Gorge

An increased demand for power as well as serious summer droughts, led to numerous studies for a possible storage reservoir at some point on the Genesee watershed. These investigations, covering a period from 1887 to 1908, were conducted chiefly by the late George W. Rafter, M. Am. Soc. C.E., under the general direction of the State Engineer and Surveyor; also by the Rochester Chamber of Commerce. The chief aim was the construction of a reservoir sufficient for an additional water supply needed for the Erie Canal east of Rochester or a larger reservoir for canal purposes and water power combined. It is interesting to note that little mention, if any, was made of these reservoirs as a means of flood control or regulation of the river flow.

In 1908 Mr. Freeman was engaged by the New York State Water Supply Commission to plan and direct its investigation. He selected for investigation the Sacandaga River near the point where it empties into the Hudson, and the Genesee River at the "Rafter site" near Portage, about 50 miles above Rochester. His study gave first consideration to the construction of a reservoir on the Genesee River for flood relief at Rochester and in the valley above it, and of a power development in connection therewith to help carry the expense. His plan proposed the construction of a dam 10 to 20 ft higher than would be required for power purposes alone, thus reserving the upper 10 to 20 ft of the reservoir, where the area is greatest, solely for flood relief.

The Sacandaga project has been completed, but nothing has been done on the Portage site for the Genesee River. Table I shows various characteristics of proposed flood storage projects in the Genesee River and its tributaries from 1908 to date.

CHANNEL IMPROVEMENT ACCOMPLISHED FOLLOWING THE FLOOD OF 1913

Nothing further was done in the matter of flood protection until the flood of March 23–27, 1913. Immediately, upon the recommendation of the mayor, ordinances were adopted for channel improvements within the city substantially as suggested by the Flood Committee of 1904. The work, extending to the spring of 1919 and costing about $1,250,000, consisted of retaining walls and repairs to existing walls; deepening of the river bed from 5 to 7 ft from the brink of the Upper Falls to a point 100 ft above Broad Street; removal of a part of the Central Avenue Dam and replacing 80 ft of it by temporary dams; also, additions to the state dock walls and retaining walls south (upstream) of the State Dam. Because of war conditions, the city was obliged to cancel the main contract and to proceed with the work at greatly increased cost. The work proposed in the original contract was therefore not fully completed.

The following related works were performed by the state in the river from

TABLE I. CHARACTERISTICS OF PROPOSED FLOOD STORAGE PROJECTS IN THE GENESEE RIVER AND TRIBUTARIES, 1908 TO DATE

Location	Drainage Area Sq Mi	Reservoir Area Sq Mi	Height of Dam: Total Ft	Height of Dam: Res. for Floods Ft	Capacity: Total Bil Cu Ft	Capacity: Res. for Floods Bil Cu Ft	Equivalent Runoff: Tributary Area Inches	Equivalent Runoff: Whole Watershed Inches	References
Portage, 1909 plans	948	13.5	145	19*	18.0	6.0	2.72	1.06	N. Y. S. Water Supply Comm., Reports 1909, 1910; est. for 19 ft additional $550,000
Portage, for floods only	948		95	95	6.0	6.0	2.72	1.06	N. Y. S. Water Supply Comm. Report, 1909; est. $2,250,000
Mount Morris for floods only	1070	3.5	120	120	6.0	6.0	2.41	1.06	N. Y. S. Water Supply Comm. Reports 1909, 1910; est. $1,200,000
Mount Morris, R. G. & E. plans	1070	5.0	240	20†	16.67	2.67	1.07	0.47	Rochester Gas & Electric Corp., various papers and plans
Twelve dams on tributaries of Genesee River above Portage	675	5.24	10-84 Ave. = 52		3.362	3.362	1.20	0.59	N. Y. S. Water Supply Comm., Pro. Report, 1908, 1909
Honeoye Lake, reservoir for additional water supply	187	12.5	60	10	9.83	3.8	8.75	0.67	Report on the Rochester Water Works by Eddy, Hazen, and Fisher, 1927
Two miles south of Dansville	90	0.46	95	95	0.503	0.503	2.4	0.09	State Water Supply Commission 1910; est. of cost, $237,000 (1910)

*First suggestion to reserve top 20 ft for floods made by Mr. Freeman, 1908. Revised and changed to 19 ft in 1909.
†Additional height of 20 ft for flood storage suggested by Mr. Fisher.

the State Dam to the Barge Canal crossing, known as the Rochester Harbor, and controlled and operated by the state:

1. In 1918 it constructed two sector gates of 54 ft each and a Mohawk-type dam of 240-ft span across the river above Court Street to replace the existing Johnson and Seymour Dam and control the Barge Canal level from Rochester to Lockport, a distance of 60 miles. This is known as the State Dam and provides a navigable stage of 512.6 ft.

2. In 1921 it excavated a channel 200 ft wide above this dam to El. 500.6 to provide for a 12-ft depth of water in the rock bed of the river where needed, for the Barge Canal Harbor.

3. In 1926 it authorized the Rochester Gas and Electric Corporation to replace the Mohawk-type dam noted in (1) with two sector dams each 110 ft long and of a height 6 in. greater than the Mohawk Dam. When completed, this dam became the property of the state. The agreement provided, among other matters, that the state license the corporation to use the surplus water from the Barge Canal, estimated at approximately 600 cu ft per sec, upon the payment of $50,000 per year, subject to readjustment at the expiration of every ten years during the 30-year term of the license.

North of (below) the upper falls, the Genesee River flows through a gorge to Lake Ontario. In this section the danger of flood damage is comparatively small. Hence the main problem within the city of Rochester is to improve the existing channel from the upper falls upstream (south) for about 4 miles to the crossing of the Barge Canal.

WHAT THE COMPREHENSIVE PLAN CONSISTS OF

As finally evolved, the plan for increasing the existing capacity of about 55,000 cu ft per sec to a safe flow of 90,000 cu ft per sec included:

1. Removal of the old permanent and temporary dams at Central Avenue and replacement by electrically operated lift gates as completed under contract with PWA in 1937.
2. Additional deepening at numerous places.
3. Additions to the height of protection walls and embankments, with removal of projections over existing walls.
4. Widening and straightening of the river channel.
5. Making the channel sides and bottom smooth.
6. Replacing the existing Andrews Street Bridge of seven spans by a bridge of three spans.
7. Replacing the existing Main Street Bridge of five spans by one of three spans. The plan included either the removal of the present pier extensions and overhead structures and buildings on both sides of the bridge; or the reconstruction of the pier extensions to correspond with the new bridge piers and to provide for a safer foundation and greater clearance for floods.

Execution of this complete plan has been subdivided into five construction sections (Fig. 1), which will be described individually.

Section 1. This section extends from the brink of the Upper Falls to a point 150 ft south of Central Avenue Bridge, a distance of about 400 ft. Work in this section was done in cooperation with the Rochester Gas and Electric Corporation. Two contracts, awarded to the same contractor, were completed in 1937.

The city's contract included the completion of river deepening at the bridge, removal of the old temporary and permanent dams together with the underlying rock, lining of the river bed between bridge piers with gunite, extension of existing bridge piers to permit the future widening of Central Avenue from 66 to 100 ft in accordance with the city plan, also to permit the corporation to replace the old dams with six Broome-type crest gates, each having an average width of 40 ft and a height of 11 ft. The gate frames are formed into rectangular tubes as "heater boxes," piped for steam to free the gates if frozen in place. A steel superstructure supports the operating mechanism, including a gantry-type trash hoist, with a grab hook to assist in clearing driftwood lodging on the gates during high water.

For Brown's Race intake, north of the west dam gate, it was necessary to install an auxiliary dam. This consists of vertical wooden stop-logs or "needles" supported at the bottom by a slot in the dam sill and at the top by a horizontal I-beam which can be tripped mechanically to collapse the dam and clear the channel during times of extreme flood flow.

The final cost of the work done by the city amounted to $238,823.91, of which $102,215.86 was a PWA grant from the Government. Furnishing and erecting the gates, overhead structure, and operating machinery, together with other incidental work done and paid for by the power company, amounted to about $100,000. The Central Avenue Dam is maintained by the Rochester Gas and Electric Corporation and operated by it in times of high water under the direction of the Commissioner of Public Works.

Section 2. The next section upstream extends from the Central Avenue Dam 1,500 ft to the north end of the pier extensions of Main Street Bridge. The work to be done includes additional deepening and concreting of the river bed where required; reconstruction and repair of river walls; raising or removal of projecting buildings within the channel lines; also replacement of the existing Andrews Street Bridge of seven spans by one of three spans.

In general the river walls are a part of existing buildings and are of old masonry structure. The outside mortar has badly deteriorated, leaving the walls in a loose and porous condition. The repairs would consist of a new reinforced concrete wall 12 in. thick, built against the old walls from the existing rock ledge up to El. 485; and a 2-in. gunite facing from El. 485 up to a line 2 to 5 ft above the expected flood crest of 90,000 cu ft per sec.

Section 3. This includes the five-span Main Street Bridge and extends from the north end to the south end of the bridge pier extensions, a total of 350 ft. The plan,

New Central Avenue Dam in Process of Construction, Showing West End of New Dam Completed
View Looking North (Downstream) from West Bank

in brief, contemplates the reconstruction of the bridge and the structures on both sides, together with pier foundations. The west abutment is to be moved 15 ft further west, giving the westerly span a clear width of 46 ft at top of piers; the west pier to be rebuilt practically in its present location; the next to be removed; the third to be rebuilt; the fourth, in turn, to be removed; and the east abutment to be rebuilt. This arrangement will provide for three spans—the west span, 46 ft; middle span, 88 ft; and east span, 78 ft. The river bed is to be deepened 2 ft to compensate for the obstruction caused by the two bridge piers.

SOUTH SIDE OF MAIN STREET BRIDGE VIEWED FROM ABOVE
Showing Rear of Existing Buildings and Encroachment on Bridge Arches

Purely as a flood-protection measure, the buildings over the river should be entirely removed. There are, however, many reasons why it would be desirable to maintain these structures if it could be done so as not to interfere with the flood flow of the river. Buildings over the river in the present location have been maintained for more than 50 years. The 1935 assessed valuation of the land and buildings involved in the rebuilding amounts to about $2,500,000.

Section 4. This section reaches from the south end of Main Street Bridge pier extensions to the State Dam, a total length of about 1,300 ft. It is proposed to deepen the river bed from about 2 ft to 8 ft; also, to make the bed and sides smooth by facing with concrete and gunite where required.

The old east river wall from the Aqueduct (Broad Street) to the Court Street Bridge has been replaced by the west building wall of the Public Library in accordance with flood control plans. The existing wall from Court Street to the State Dam, forming the foundation for the Lehigh Valley Railroad trestle, has also been improved in height by a temporary concrete construction of 4 to 5 ft. Both walls were completed in 1936.

On the other side it is proposed to straighten the river channel by constructing an additional arch at the westerly end of the Court Street Bridge. For this purpose the old west river wall south of Court Street to the State Dam has been removed and a new concrete wall in the new location constructed as a WPA project. North of Court Street it is proposed to construct a new river wall from the new abutment to the westerly pier of the Broad Street Bridge, which wall may form the foundation of a new municipal building.

Section 5. This section is a part of the Barge Canal and is controlled, operated, and maintained by the state. Work on this upper section, extending from the State Dam south to the Barge Canal crossing, a distance of about $3^1/_4$ miles, includes additions to existing walls and dikes and construction of new walls, dikes, and riprap where required. The existing walls and docks built by, and under the control of the state are in bad condition and should be restored before any additions are made. Portions of the walls used for dock purposes should be provided with an effective temporary flashboard structure which should be kept in readiness at all times for immediate protection against floods.

Taken as a whole, the features remaining to be undertaken constitute a $4,000,000 project. In any event, channel improvements substantially as herein outlined, should be constructed even if reservoirs are built on the watershed.

As justification for such an expenditure, we made a computation of the direct damage from a flood of 90,000 cu ft per sec through the city, under existing conditions in the channel and watershed. For such an estimate we had no actual data in the territory on which to base calculations. After a careful study of the methods used in various flooded districts, that of the Army Engineers for a comprehensive flood control plan, including reservoirs and dikes in the Connecticut Valley, as described in the November 1937 issue of CIVIL ENGINEERING, was adopted.

Referring first to the direct damages, the total area affected in the city is about 5.17 sq miles, or 3,810 acres. This area may be divided into two general subdivisions depending upon the amount of flooding. The first section, where the flooding would be greatest, contains an area of 1.29 sq miles and had an assessed valuation in 1936 of $81,000,000 (considered actual value). In this section, including the covered portion of the subway (Broad Street), which would be entirely filled and badly damaged, we estimate the amount of direct damage at $7^1/_2$ per cent of the assessed valuation, or $6,075,000. In the remaining portion of the flooded territory, having an area of 3.88 sq miles and a valuation of $72,000,000, the direct damage is estimated at $4^1/_2$ per cent of the assessed valuation, or $3,340,000. The total direct damage would then be $9,315,000. The indirect damages may be as much more.

A third item, loss in market value in the flooded district, may be as much as 10 per cent of the assessed valuation, but would be largely, if not altogether, restored by the improvements.

This estimated preventable direct damage is entirely within the city of Rochester. Some damage would also occur in the town of Brighton. The undertaking of the proposed expenditures for flood relief is thus seen to be amply warranted.

The main facts, assumptions, and conclusions leading to the adoption of a probable future flood of 90,000 cu ft per sec may be found in a report made by the writers in collaboration with Mr. Fréeman, acting as advisory consulting engineer. The report, entitled *Flood Conditions in the Genesee River,* including specific relations to a civic center, together with a digest of former reports and a reference to the large floods of 1935, 1936, and 1937 in the eastern part of the United States, was made to the Hon. Harold W. Baker, city manager, and published by him in 1937. This and other reports dealing with the Genesee flood problem at Rochester, together with a copy of the comprehensive paper from which the foregoing has been abridged, have been filed in the Engineering Societies Library for record and reference.

Improving Foundation Rock for Dams

A Brief Review of Grouting Technique

By James B. Hays

Member American Society of Civil Engineers

Construction Engineer, Chickamauga Dam, Chattanooga, Tenn.

There are two purposes, says Mr. Hays, in grouting a dam foundation. The first is to provide a cutoff curtain; the second is to assure a uniform deformation over the loaded area. Few if any fixed rules can be set for grouting procedure, but "with a complete geologic record, any site can be successfully treated." The author discusses materials, methods, and equipment, giving special attention to a number of recent developments—such as the gage to measure uplift during grouting, and the by-pass circulation of grout to avoid clogging of feed lines. The present article is an abridgment of Mr. Hays' paper on the Power Division program at the 1939 Spring Meeting in Chattanooga.

Treatment of the foundations of a dam is the art of remaking the geology of the site to fit the requirements of a safe structure. Largely it is a problem of grouting; and the purpose of this article is to present a brief review of current practice in that field.

In general, there are two objectives in grouting a dam foundation. The first, and often the only one considered, is the elimination or reduction of leakage. This is accomplished by drilling and grouting one or more lines of holes in the foundation at or near the upstream face to seal off all seams or cavities. A perfect job of grouting the cutoff curtain would produce a condition of no leakage and no uplift above tailwater elevation. Of course, since it is not safe to assume that the job will be perfect, certain allowances are made in the design of the dam and, in addition, drainage is depended on to some extent.

If only cutoff grouting is done, it is possible under certain conditions that the relatively solid zone of grouted rock under the upstream toe of the dam may deflect less under load than the remainder of the area. Thus, in the case of storage reservoirs that are alternately filled and emptied, a slow rocking movement of the dam may result. In any case, there may be undesirable effects on the distribution of stress in the foundation.

The second objective of grouting, therefore, is to consolidate the rock over a part or all of the foundation area in order to assure a uniform deformation that will approach, as near as possible, the deformation assumed in the design.

Successful foundation treatment requires a thorough knowledge of the geology of the site, including the location and extent of all faults, folds, and other weaknesses in the rock. Therefore it is essential that a thorough exploration of the site be made. Accuracy in the data obtained is of prime importance in determining the procedure to be followed, and will eliminate many unforeseen and costly "extras." A competent geologist, who understands the engineering problems involved and who can properly present them, can be of great assistance and should in many instances be a regular part of the field organization.

With a complete geologic record, any site can be successfully treated. The main problem is to get a thorough job at the lowest cost.

Many different materials are employed in grouting, and each has its own use. Only one, however, has been found satisfactory for grouting under masonry dams, and that is cement. Certain admixtures, such as hydraulic lime, can be used to improve the workability of the grout and allow it to work into fine seams. (This can also be accomplished by rescreening the cement to remove the coarser particles.) Other admixtures, like calcium chloride or lumnite cement, are used to speed, and others to retard, the setting as conditions require; and small admixtures of china clay have been used to increase the resistance of the grout to erosion by water prior to final set.

Many grouting specifications have allowed the use of mixtures of sand and cement, especially where the required quantities of grout are large. If it is intended merely to fill large openings and to neglect small seams, and if there is no concern as to the permeability of the grout, the addition of sand is satisfactory. Otherwise, a safer method of filling large cavities at a minimum of expenditure for cement is to drill large holes directly into them and backfill with concrete (if this can be done in the dry), following which the usual grouting with cement is done.

In a grouting mixture of sand and cement, the cement serves as a lubricant to convey the sand. Once in the cavity, the sand settles to the bottom and the cement comes to the top, the rate and degree of segregation depending on the relative quantity of water used in the grout and the presence of water in the cavity. In this method of grouting, small seams "plug off" near the drill hole or cavity wall and are not grouted. If the high strength of cement grout is not required, bentonite may be added in small quantities to hold fine sand in suspension. This material also lubricates the mix so that it can be handled through grout pumps without plugging or excessive wear.

Rock flour, of a fineness less than 100 mesh, is often available at locations where the concrete aggregate is manufactured, and should be considered for use in grouting in lieu of sand. Tests should be made to determine whether or not it segregates from the grouting mixture when poured into a container filled with water.

Location and Spacing of Holes

In cutoff grouting, the holes are usually drilled along the upstream face of the dam, on fairly wide spacing for the primary treatment. This spacing may vary from 4 or 5 ft to 20 ft, depending on the tightness of the rock. The need for washing out the seams between holes depends on conditions. If they are filled with mud or sand, a thorough washing job should be done. In any case, water should be pumped into the holes in advance of grouting, to determine the consistency of mix to be used and to locate the leaks that should be sealed in advance to avoid excessive loss of cement. Secondary drilling and grouting should follow the primary treatment, with the holes placed midway between those in the first set.

In solid, massively bedded rock this second grouting may be sufficient. In thin-bedded or broken irregular

rock, additional grouting is generally required. Additional drilling and grouting, each time reducing the spacing between holes, should be done until a satisfactory condition is indicated by the small amount of grout finally taken by the last set of holes. Rock with vertical joints may call for holes drilled in an inclined direction to intersect the joints. In such cases the primary set may be inclined in one direction and the secondary line sloped opposite to the first, thus producing a diamond shaped or criss-cross pattern in a vertical plane parallel to the axis of the dam.

GROUT MIXING AND PUMPING UNIT ON SKIDS
Two Pumps Are Installed in Parallel, for Alternate Use

In the case of a high dam, grouting must be carried so deep and requires such high pressures that the work is generally done in "stages." Shallow drilling and grouting is done to solidify the upper zone of rock, from 20 to 50 ft thick, in order that high pressures can be applied at lower levels. For extremely high pressures, a wider zone than that covered by a single line of holes should be grouted in the first stage.

An excellent and safe method of stage grouting is to drill to a shallow depth first and grout with low pressure; then clean out the hole, drill to a greater depth, and grout with higher pressures. This can be repeated for as many stages as desired. In this way the high pressure is applied to the lower section and is generally not effective in the upper section. If, due to the condition of the rock, it is felt that there is danger of splitting seams in the upper zone, "packers" on the end of the grout pipe can be used at the bottom of the section previously grouted. This will prevent any pressure from being applied on the upper sections of the hole. Low pressures for shallow holes are generally assumed to range from 25 to 100 lb, medium high pressures from 100 to 500 lb, and high pressures from 500 to 1,000 lb or over.

Thin-bedded horizontal layers of rock are quite easily lifted under rather low pressures. This calls for closer spacing of holes and lower grout pressures, since under these conditions very little spread of the grout can be expected in thin seams. Frequently holes even shallower than 20 ft are advisable for the first stage.

UPLIFT INDICATORS WARN OF DANGER

Where there is danger that uplift may occur during grouting, uplift indicators or gages are sometimes used. Satisfactory devices of this type have been constructed as follows: A hole a few feet deeper than the lowest seam to be grouted is drilled, and a pipe casing is grouted in at the bottom. Inside the casing a rod is installed, also grouted in at the bottom, and projecting a few inches from the top of the hole. A yoke, consisting of a steel bar bent to a U shape, is secured in the top layer of surface rock directly over the vertical rod. It is set so that two contact points, one on the bar and one on the yoke, barely touch. Any uplift during grouting can be observed by the opening between these points. The dip of the rock bedding determines whether uplift gages are needed and where they should be located. With nearly vertical bedding, the movement of rock during grouting would be more in a lateral direction than vertically, and uplift gages would be of very little value.

SEALING SURFACE SEAMS AND CRACKS

If the rock is inclined, many surface leaks may be found in the washing operations. These can be patched by driving thin wooden wedges into the crack, by calking with lead wool or oakum, or by mortar or concrete. Calking, however, is often unsatisfactory because of the progressive opening of the joint. Lead wool lacks resiliency and does not expand if the seam is opened by grout pressure; hence recalking is required to prevent blowouts. Under some conditions, such as in relatively flat bedded rock where uplift must be particularly guarded against, the use of lead wool might be desirable, for leakage from a crack thus calked would be an indication of movement.

Cracks or seams should not be completely sealed off by calking prior to grouting. Rather, leaks should be permitted at predetermined points along the cracks in order to be certain that the seams have been thoroughly filled. Each opening should be closed as soon as thick grout appears, and grouting continued until the condition required by the specifications has been reached. If the area to be grouted has been covered with concrete, pipes should be provided, or holes drilled into the seam, to act as weep holes. This will allow the escape of air or water and permit complete filling of the seams by capping each pipe as soon as thick grout starts to flow.

Whether grouting should be done after a certain height of concrete has been placed, or directly from the rock surface, depends in part on the rock and its structure. Most shales and thin horizontally bedded rocks require some cover. Under any conditions, high-pressure grout holes are often drilled after concrete has been placed, and provision should be made for them by installing pipes at the time of pouring. These pipes serve partly to reduce the cost and speed up the work of drilling, but their main purpose is to prevent high-pressure grout from cracking the concrete. When they have not been provided initially, arrangements should be made to insert them in the drilled holes and to use packers below the concrete to prevent the pressure from being applied directly to the structure.

Consolidation grouting over the entire area of the foundation should be done by following an interlocking pattern in so far as possible. In connecting one grouting operation with an area already grouted, the adequacy of washing in the zone between the two is questionable, and the only way to overcome this uncertainty is to stagger the joints as much as construction operations will permit.

Ordinarily consolidation grouting is not carried as deep as the cutoff curtain. Practice varies, depending on the rock, and probably such grouting generally extends down from 20 to 50 ft. The depth and extent of seams in the rock is the governing factor, the objective being to solidify the foundation to create an even deformation and properly distribute the load. In flat bedded rock, consolidation grouting is likely to produce more uplift than cutoff grouting, since seams are washed out over a large area rather than along a narrow line or zone.

Pressures in all types of grouting vary with the consistency of the grout mix as well as with the depth of the seams and with other geologic conditions. Thick grout has a greater pressure drop in a unit distance than thin grout. There is a considerable drop in pressure where the grout is forced from the hole into thin seams. Here a small orifice exists, with often a relatively large space to be filled beyond it. The grout may flow into this outer space under very low pressure for a long enough period to cause most of it to have an initial or even final set before pressure can be applied.

PRESSURES, GROUT PROPORTIONS, AND EQUIPMENT

No fixed rule for pressures can be given. In large massive layers, holes 50 ft deep can be given pressures of 100 lb with safety. In thin-bedded horizontal strata, 25 lb may produce uplift in holes of the same depth. In deep high-pressure grouting for the cutoff curtain, where shallow grouting has previously been done, progressively higher pressures can generally be used. If 100 lb was the safe limit for 50-ft holes, 100-ft holes should generally stand 300 lb. Below 100 ft, in a section previously grouted to that depth, 500 lb might be used. Geologic conditions will govern. Pressures as high as 1,000 lb can be used below 100 ft in good rock, whereas in poor rock 300 lb might be the limit.

Mixtures have usually been based on the water-cement ratio by volume, although there is no reason why they should not be based on weight. Mixtures will vary from 5 parts of water to one part of cement by volume for a thin mix, to be used under tight conditions, to 0.6 part water to one part of cement for a thick mix for more or less open seams. The thickest grout produces the least shrinkage and should be used as conditions permit, bearing in mind that a large total quantity of grout indicates a maximum filling of voids. The highest practical pressures should be used in order to force all surplus water out of the grout and thus lower shrinkage.

The most satisfactory equipment for injecting grout is a pump of the double-acting duplex reciprocating type. This gives a steady flow, which is more effective, especially in fine seams, than the intermittent flow produced by compressed-air equipment, and the operator has a more even control of the pressure. Unless extreme care is used, the air-pressure method is apt to force air into the seams if the grout flows faster than expected. Moreover, in some types of air-injection tanks the cement in a thin mix will settle to the bottom and plug the outlet, the pipe line, or the hole, before the job is really completed.

The pump should be one especially fitted for grouting work. Cylinders should have renewable liners that can be conveniently replaced. The piston should be of rubber; valve facings should also be of rubber, of stock similar to automobile tire tread material. A duplicate set of pumps should be so arranged that in an emergency the second pump can be put into operation without delay.

For small jobs the grout can be mixed in a small concrete mixer, but ordinarily a special grout mixer will be found more economical. The grout should be discharged from the mixer through a screen (to remove hard lumps and castings from the grinding mills), and into a tank or sump from which it can be pumped. The sump should have an agitator, consisting of a blade or paddle, running fast enough to keep the grout thoroughly mixed.

The most efficient piping system provides two lines to the hole; one carries the grout from the pump and the second is a return line discharging into the sump. At the hole the pipe connections are as follows: The feed or supply line connects to one end of a tee; the side outlet on the tee, facing downward, feeds the hole through a valve, below which a gage is attached; and the other end of the tee connects to the return line through a second valve. Since the grout is constantly circulated through the feed line and back to the pump through the return, the tendency to plug the supply pipe is eliminated.

A Portable Grout Mixing and Pumping Plant
The Unit Includes Mixer, Water Meter, Sump Tank, Pump, and Connections for Air, Water, and Grout Lines

(Thin grout, required on tight holes, is as apt to plug a single-line system as thick grout, since the movement is slow and settlement of cement occurs quite rapidly.) By proper operation of the two valves in accordance with the gage indication, the pressure at the hole can be accurately controlled. The rate of grout consumption, as shown by the quantity of return flow or by the number of batches used per hour, indicates to the inspector whether he has the proper mix.

GENERAL GROUTING PROCEDURE

Few if any fixed rules can be set for grouting procedure. The engineer or inspector must have all available information in advance. He must know the design requirements and something of the structural geology. The more accurate the information, the more thoroughly can the job be done. In general, it is best to start each hole with a thin mix (the consistency depending on the results of the water test), and to increase the consistency with each batch mixed, at the discretion of the inspector, until about 75 per cent of the final desired pressure is reached, in order that grout be forced into as many seams as possible at the same time. There will be times when the maximum pressure is reached, which is not objectionable if no uplift occurs. Higher pressures speed up the work and do a more thorough job if they can be used. Where limited to a definite maximum pressure, the working range should be between 75 and 100 per cent thereof.

The pumping rate should be kept fairly constant. Ordinarily a normal output rate for the pump is used, to allow some flexibility. Naturally this rate varies with the size of pump. The pump size should be selected beforehand, and should not be too small. In deep or high-pressure holes the consistency of the grout should be reduced if the pressure rises, even to the extent of pumping water with no cement into the hole for short periods. This often clears up the passages leading to small tight seams, and grouting can continue, the process being repeated until water is refused. In the case of shallow grouting, driving of additional holes is not so costly and is often preferred to pumping water to open fine seams.

Specifications often state that grouting of a hole should continue until the quantity taken reaches a certain pre-

determined rate. They should also indicate the consistency and the pressure.

Where it is not necessary to drill a set of holes to facilitate washing of seams, holes drilled and grouted one at a time will give the best result. If a set of holes is required, grouting should still be done consecutively, beginning with the hole penetrating the largest seam. Hooking up all holes to one grout header will cause most of the grout to flow to the hole having the least resistance, and if this hole requires thick grout, others that would take an appreciable quantity of a thinner mix will quickly plug up and become "lost" as far as effective grouting is concerned. Grout applied at one hole will of course spread out and leak into others, and if a thick mix is being used it will probably plug or seal off small seams in the leaking holes, especially when it is necessary to cap them to hold the pressure. Holes that leak grout may be jetted out while the cement is still soft, and regrouted. Although the efficiency of this procedure is not all that could be desired, it is the best that can be done under the circumstances.

Leakage into neighboring holes indicates the travel of the grout and whether or not the seams are being filled at all points. From the record of the first grouting an intelligent procedure can be worked out for the secondary system. Theoretically, the largest seams are taken care of by the primary washing and grouting, and the secondary treatment reaches the next smaller seams. Further treatment may be required, depending on the results of the secondary operation.

One of the most troublesome problems in grouting is the treatment of a seam or cavity in which flowing water is encountered. If not too large, the seam can be plugged by the addition of sawdust to the grout. (Where the hole is of sufficient size, shavings are used.) Sawdust will float out of the mixture and choke up reasonably small seams. As soon as the leak has been stopped, grouting with neat cement mixes of thick consistency should follow, using care not to apply high pressure until a partial set of the cement has taken place.

If an area of the foundation has springs indicating an artesian condition, exploration should be undertaken to locate the direction of flow. The pressure can be relieved by drilling a hole near where the flow enters the foundation area and allowing the water to escape more readily by direct relief or by pumping. The rest of the area can then be grouted and, when all completed except the relief hole, this may be similarly treated. There may be conditions where it is advisable to permit the relief hole to continue to flow.

PROPER LOCATION OF DRAINAGE HOLES

After the grout cutoff curtain has been completed, holes are drilled for drainage. Too often the drainage holes are drilled so close to the cutoff line that many of them penetrate rock that has been grouted, and consequently are ineffective in relieving uplift pressures. For example, where cutoff grouting is done from a gallery near the upstream face of the dam, the practice has often been to drill the grout holes vertically and then, after grouting, to drill the drainage holes, also vertically, and only a few feet downstream. It would be an improvement to drill the grout holes at an angle of, say, 30 deg from the axis in an upstream direction, and then to drill the drain holes at an angle of from 30 to 45 deg downstream from the axis. In this way the drain holes would be quite certain to penetrate an ungrouted zone, provided they were of sufficient depth. If the entire area of the foundation is grouted, drainage must be more thoroughly done, since it must be assumed that all seams to the depth of the grouting have been filled. Drainage should generally be considered over the entire base of the dam rather than only at a point immediately downstream from the grout curtain.

OTHER GROUTING MATERIALS; CHEMICAL GROUTING

As previously mentioned, numerous materials other than cement are useful in a variety of grouting problems. Stabilized clay grouts, for example, are satisfactory and economical for certain purposes, and asphalt and pitch are often effective in seams with flowing water. Because of the low strength of these grouts, they are not usually considered for use under masonry dams.

Chemical grouting is useful in the treatment or solidification of sand and gravel, conditions which are not usually encountered in the foundations of masonry dams. The materials to be grouted must have sufficient pore space (or fine cracks) to permit the chemicals to penetrate. Clays, silts, clayey sands and the like cannot be treated. The method most generally known consists in pumping into the area to be grouted a sodium silicate solution, commonly known as water-glass, and following it with a solution of calcium chloride. The resultant chemical reaction produces a solid. This method was developed in Germany and is patented both there and in the United States. At least one company in this country is licensed to use it.

Other processes include one using sodium silicate and aluminum sulfate; another using silicic acid and a gas which causes the mass to gel; and a third (invented and patented by the late Lars R. Jorgensen, M. Am. Soc. C.E.), which utilizes a combination of sodium silicate, calcium chloride, and a gas (carbon dioxide).

Chemical grouting has not been used to any great extent in the United States, largely because of the complications of the process and the uncertainty in the minds of many engineers as to the results. Mr. Jorgensen used chemical grouting methods in a number of instances along the Pacific Coast (see his article, "Solidifying Gravel, Sand, and Weak Rock," in *Western Construction News*, November 10, 1931, page 591), but stated that they had a field of their own and did not attempt to compete with cement grouting where such methods were suitable.

The chemical reaction is quite rapid. No rules can be set up for the quantity of sodium silicate that is pumped in before the addition of the calcium chloride. Usually it is based on the maximum allowable pressure that can be used. The first chemical is forced in at low pressure. The second chemical then requires a higher pressure in order to force in a sufficient amount to combine with all of the first chemical. In one process, the two chemicals are combined at the entrance to the grout hole and forced in under high pressure.

Recently a new process has been developed that provides for mixing the chemicals in such a way as to control the time of set. This will no doubt reduce the complications and uncertainty inherent in the two-stage methods.

The cost of chemical grouting covers about the same range as cement grouting and depends on the volume of materials used. In addition, there is the cost of using a patented process.

In connection with chemical grouting, it may be noted that sodium silicate added to cement grout may be useful in stopping leaks or flows of such volume and velocity as to wash out ordinary cement mixtures. The silicate, combining with the free lime in the cement, forms a thick plastic mass that sets quickly, and should be introduced into the cement grout in the hole at or near the leak to be grouted.

ENGINEERS' NOTEBOOK

This department, designed to contain ingenious suggestions and practical data from engineers both young and old, should prove helpful in the solution of many troublesome problems. Reprints of the complete department, $8^1/_2$ by 11 in., suitable for binding in loose-leaf style, are available each month at 15 cents a copy.

A New Method of Presenting Data on Fluid Flow in Pipes

By A. A. Kalinske

Research Engineer, Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, Iowa

The presentation of fluid friction data for pipes by plotting the friction factor, f, against Reynolds' number, **R**, has been accepted as the ideal method of correlating such data. The factor f is the term occurring in the ordinary fluid friction equation, $h = \frac{fL}{D}\frac{V^2}{2g}$; and $\mathbf{R} = \frac{DV}{\nu}$, where ν is the kinematic viscosity. By methods of dimensional analysis it can be shown that for hydraulically smooth pipes f should be a function of **R**. In other words, if f is plotted against **R** for any size of pipe all the points will fall on one smooth curve. Experimental data on very smooth pipes verify this fact.

For pipes that are not hydraulically smooth—that is, for many commercial pipes—a plotting of f and **R** for various sizes will not give one smooth curve; instead, the data for different sizes of pipes will tend to fall on separate curves. The rougher the pipes are, the more distinctly will the data fall on separate curves for each size of pipe. This means that another variable is affecting the friction loss data, which is not taken into account by a simple relation between f and **R**.

Since at present no method of analysis is available that will completely define any given roughness, it is necessary to resort to various approximations which will give some measure of roughness and thus permit a better correlation of fluid flow data for commercial rough pipes.

Prandtl and Von Karman have shown by an analysis of the inner mechanism of turbulence, that for hydraulically smooth pipes $\frac{1}{\sqrt{f}}$ should be a function of the quantity $\mathbf{R}\sqrt{f}$, which we can call K. Note that K is really another dimensionless quantity.

$$K = \mathbf{R}\sqrt{f} = \frac{h_f^{1/2}D^{3/2}\sqrt{2g}}{\nu} \qquad [1]$$

in which h_f is head loss per unit length of pipe. The term K is quite useful, as it can be calculated if the pressure loss is known but not the velocity. To calculate the Reynolds number the velocity must be known. The fact that $\frac{1}{\sqrt{f}}$ should be a function of K for smooth pipes can also be shown by dimensional analysis.

If $\frac{1}{\sqrt{f}}$ is plotted against $\log K$, for smooth pipes of any size, there results a straight line having the equation:

$$\frac{1}{\sqrt{f}} = 2\log K - 0.80 \qquad [2]$$

(All logarithms in this paper are to the base10.) Prandtl and others have shown that this equation holds for all values of R in the region of turbulent flow.

For flow in extremely rough pipes, experiments indicate that above a certain value of **R**, f is independent of **R**, or in other words, viscosity has no effect on the value of f. This fact is also true for pipes that are only moderately rough, at the higher values of the Reynolds number.

Using artificially roughened surfaces, Nikuradse, Schlichting, and others have shown, for conditions when viscosity has no influence on the value of the friction factor f, that f is dependent only on the relative roughness, k/r, where k is the height of the roughness projections and r the radius of the pipe. Various analyses indicate that $\frac{1}{\sqrt{f}}$ should be a linear function of $\log(r/k)$, and experimental data on artificially roughened surfaces verify this fact. Nikuradse, using sand-roughened pipes, obtained the following equation:

$$\frac{1}{\sqrt{f}} = 2\log\left(\frac{r}{k}\right) + 1.74 \qquad [3]$$

Since k does not completely define roughness, the factor 1.74 applies only to the type of roughness used by Nikuradse. Other types of roughness would give different values for this factor. However, theoretical consideration indicates that the coefficient 2 does not change with variation in roughness and is a sort of "universal constant." If this is true, then a plotting of $\frac{1}{\sqrt{f}}$ against $\log r$, for values of f which are independent of **R**, should give a straight line having a slope of 2.

Whether the foregoing analysis applies to natural roughness has never, to the knowledge of the author, been verified. The reason for this is that reliable experimental data from which values of f can be obtained for high Reynolds numbers, for a series of commercial pipes of the same material, is quite meager. However, friction loss data recently published by Kessler for wrought-iron pipe, including all sizes from $^1/_4$ to 8 in., seem to be fairly satisfactory for an analysis of the sort just described, since for most of the sizes the values of f were obtained at values of **R** for which f approached a constant value. (See "Experimental Investigations of Friction Losses in Wrought-Iron Pipe," L. H. Kessler, Assoc. M. Am. Soc. C.E., Bulletin No. 82, University of Wisconsin Engineering Experiment Station, 1935.)

In Fig. 1 is shown a plotting of $\frac{1}{\sqrt{f}}$ against $\log r$ from Kessler's data, using values of f which for any size of pipe were independent of Reynolds' number. The straight line that best fits the data has a slope of 2, and its equation is:

$$\frac{1}{\sqrt{f}} = 2\log r + 9.25 \qquad [4]$$

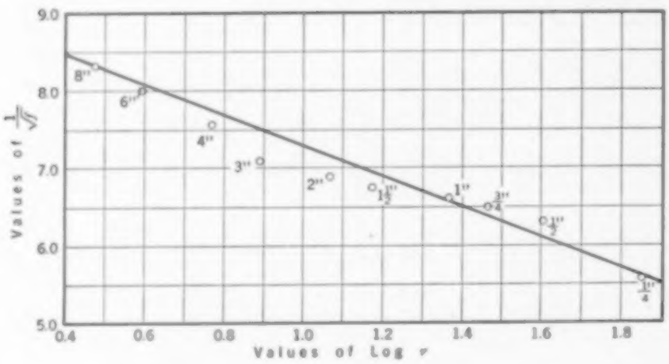


FIG. 1. WROUGHT-IRON PIPE FRICTION DATA FOR CONDITION WHEN f BECOMES INDEPENDENT OF REYNOLDS' NUMBER

Compiled from Kessler's Experiments with Water, Using Eq. 4

Thus the rough pipe equation is verified for a natural surface. From Eq. 4 the value of equivalent Nikuradse sand roughness can be determined by letting $9.25 = -2 \log k + 1.74$. The computed equivalent value of k is 0.000178 ft.

The problem now remaining is how all experimental data for such commercial pipe as wrought iron can be correlated for all values of **R** so that the data for all sizes will follow one smooth curve. In other words, what can be done in the region where both viscosity and pipe roughness affect the value of f? This is the region in which most practical pipe computations fall.

Colebrook and White ("Experiments with Fluid Friction in Roughened Pipes," *Proceedings* of Royal Society of London, Vol. 161A, 1937, page 367) suggest a method of correlating data in the region where viscosity and roughness affect the value of f. By dimensional analysis and other considerations which we need not go into here, it can be shown that:

$$\frac{1}{\sqrt{f}} = \phi \log\left(\frac{k}{r}\mathbf{R}\sqrt{f}\right) + 2\log\left(\frac{r}{k}\right) + C \quad \ldots\ldots [5]$$

Here ϕ means "a function of," and C is a constant.

This is the general equation relating f, r, **R**, and our roughness measure, k. For smooth pipes the function sign in front of the first term on the right side of Eq. 5 is replaced by the coefficient 2, and then the two terms on the right combine into $\log \mathbf{R}\sqrt{f}$, and C becomes -0.8. For conditions when f does not depend on viscosity, the term $\log\left(\frac{k}{r}\mathbf{R}\sqrt{f}\right)$ drops out, and we have the rough-pipe equation (Eq. 3).

If Eq. 5 is truly a general relation, then if $\left(\frac{1}{\sqrt{f}} - 2\log\frac{r}{k}\right)$ is plotted against $\log\left(\frac{k}{r}\mathbf{R}\sqrt{f}\right)$, all the data for all sizes of pipe of one material should fall on a single curve. This was done for Kessler's data on wrought-iron pipe and the plotting is shown in Fig. 2. For convenience r was replaced with D, and the coefficient 3.7 was inserted in the ordinate term $\left(\frac{1}{\sqrt{f}} - 2\log\frac{3.7D}{k}\right)$, so that this term will approach zero when the friction factor f becomes independent of viscosity. Note that this ordinate term is really equivalent to Eq. 3. The smooth pipe Eq. 2 is shown in Fig. 2, to give an idea of how the wrought-iron pipe data compare with those for hydraulically smooth pipes. Each plotted point is the center of gravity of a number of experimental points.

Though the data show appreciable dispersion, it is believed that the law represented by Eq. 5 is verified. Kessler's data were obtained on long lengths of pipes installed with couplings, and the different sizes did not all have exactly the same surface conditions. Also, the experimental work was done by students over a period of years, and it is extremely unlikely that uniform test conditions were maintained at all times. Everything considered, the correlation of data shown by the plotting in Fig. 2 seems to be quite significant.

Such plotting of friction-loss data permits the obtaining of the value of f from one curve for all values of Reynolds' number and all diameters of any pipe of a given material. A series of curves of this type for different pipe materials would certainly be of interest and of practical value. How the value of f varies for different pipe materials, for different fluids, for various sizes of pipe, and at any value of velocity can be seen at a glance, or after a few simple calculations, from a plotting of data such as is given in Fig. 2.

Although the author does not wish to imply that this analysis is the ultimate as far as fluid friction in pipes is concerned, it is nevertheless another step in our increasing knowledge of the complicated problem of turbulent flow in commercial conduits. It is hoped that further data will be uncovered from old experiments, or obtained from new experiments on commercial rough pipes to permit making analyses similar to the foregoing on various other piping materials.

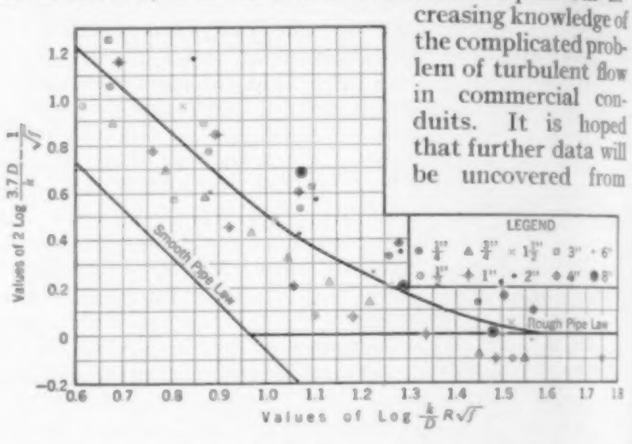


FIG. 2. WROUGHT-IRON PIPE FRICTION DATA, FOR FULL RANGE OF VALUES OF f

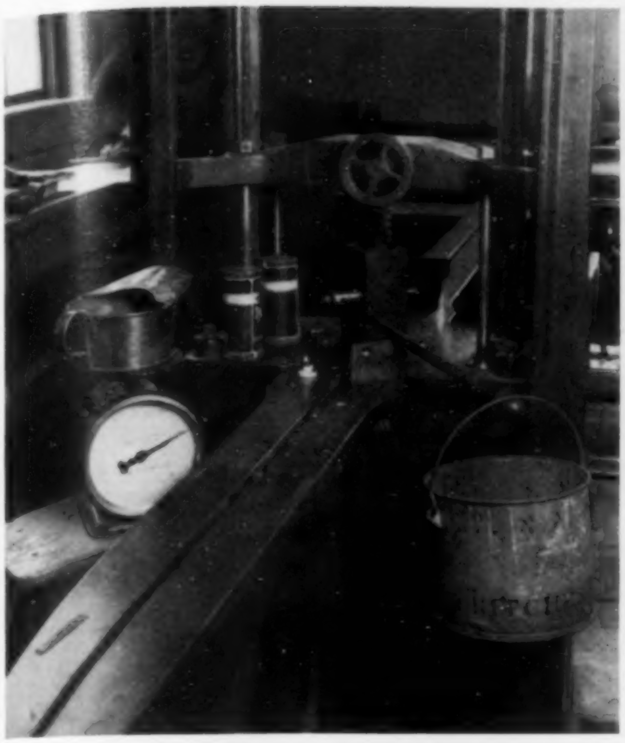
Compiled from Kessler's Experiments with Water, Using Eq. 5 (Modified)

Calibration Characteristics of Friction Bearing Under Load Tests

By WALTER J. GRAY, JUN. AM. SOC. C.E.

TOPOGRAPHICAL DRAFTSMAN, BUREAU OF SEWERS AND HIGHWAYS, OFFICE OF THE PRESIDENT, BOROUGH OF MANHATTAN, NEW YORK, N.Y.

THE increased interest in the analysis of load distribution through various media has accelerated the development of measuring instruments for the experimental determination of load action. In each case the requirements of the particular problem under investigation determine the type of instrument to be used. Many such devices are dependent for their operation on basic friction principles, two recent examples being the friction ribbon tape used by the Iowa State College Engineering Experiment Station at Ames, and the pressure cell developed at the University of Illinois.



FRICTION BEARING UNDER TEST

In general, friction devices show characteristics that require numerous readings and averages before they may be considered to act with a known degree of precision. Further, fundamental knowledge of the physical and chemical properties of surfaces in contact is not very clear. In view of the lack of information concerning observed characteristics of friction devices, the author is prompted to present the following data obtained in the calibration of a friction bearing which was intended for use as a load measuring instrument.

The device was tested in the Iowa State College Engineering Experiment Station as part of an investigation of measuring devices for use in determining the distribution of the end reaction of a test bridge slab. It was proposed to insert several such measuring instruments between the concrete test slab and the supporting pier, the instruments thus recording the end reaction distribution for a simple-span slab. However, it was concluded from the data presented here that a friction bearing did not offer sufficient accuracy and reliability for the work required; hence instruments operating on other than friction principles were developed.

As the friction bearing was not tested beyond the calibration stage, the data obtained are limited. They should, however, serve to indicate the action of this type of device, and may clarify understanding of the operation of other friction instruments.

The friction bearing tested, as shown in the accompanying photograph, consisted of a polished steel shaft which rotated in split bronze bushings, the latter keyed to cast-iron cover blocks. The torque necessary to overcome static friction was applied by adding shot to a bucket placed eccentrically on a lever rod connected to the shaft. The bearing was calibrated by applying load in a mechanical testing machine, the load passing through a spring to insure accurate application. Readings of torque were taken against applied load increments of 100 lb, the maximum applied load being 1,200 lb. Ten such readings at each load comprised a run, four runs being taken over a period of about a week. The bearing was reassembled and cleaned with gasoline preparatory to each run.

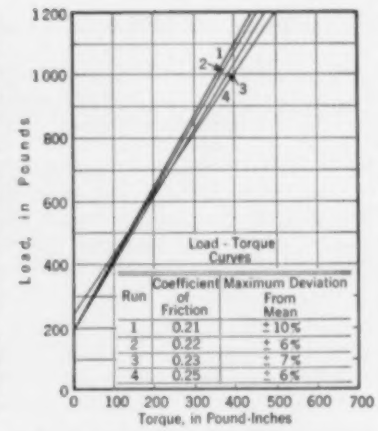


FIG. 1. RESULTS OF FOUR TESTS OF FRICTION BEARING DESIGNED FOR USE AS LOAD-MEASURING DEVICE

Note Progressive Increase in Coefficient of Friction

"Neat lines" through the average results of each of the four test runs are shown in Fig. 1, while in Fig. 2 the 10 individual readings of Run 1 are plotted to show the characteristic "band spread." Similar bands were obtained when Runs 2, 3, and 4 were plotted in the same manner.

From Fig. 1 it is noted that as the tests continued a corresponding rotation of the neat line occurred, indicating a change in the coefficient of friction from 0.21 to 0.25. The typical band spread characteristic shown in Fig. 2 indicated a maximum deviation from the mean of about plus or minus 10 per cent for Run 1, this deviation being independent of the applied load, as is shown by the intersection of the "band limit lines" and the neat line at a common origin. For Runs 2, 3, and 4, the maximum deviation from the mean decreased to plus or minus 6, 7, and 6 per cent, respectively.

Similar tests were conducted on friction bearings consisting of polished steel shafts rotating in bushings of brass, babbit metal, and oilite. In each case a neat line was readily drawn through the averages, and plotting of the individual readings showed the characteristic dispersion. For these metals the maximum deviations from the mean were respectively plus or minus 14, 11, and 10 per cent. The corresponding coefficients of friction were 0.28, 0.59, and 0.22.

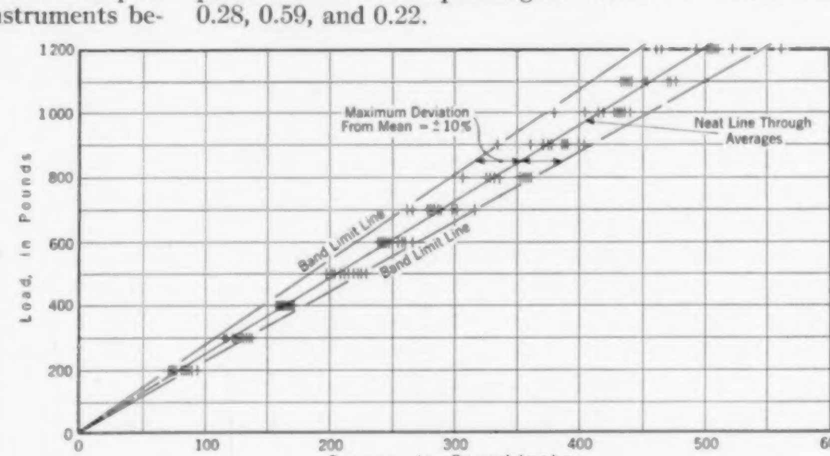


FIG. 2. DETAILED DATA FROM RUN NO. 1, SHOWING "BAND SPREAD"

This article is a brief abstract of a thesis, "Proposed Measuring Devices for the Determination of End Reaction Distribution in Floor Slabs," submitted to the Iowa State College in June 1938 in partial fulfillment of requirements for the degree of master of science in structural engineering.

Characteristics of Sawdust Concrete

By John C. Sprague, Assoc. M. Am. Soc. C.E.

Assistant Engineer, Chief of Concrete and Soils Section, U. S. Engineer Office, Huntington, W.Va.

SAWDUST concrete—a mixture of cement, sand, and sawdust—occasionally finds use in roof construction. Because of the paucity of data on this material, the test results presented here may be of interest. The investigation was conducted by the District Testing Laboratory of the Huntington District, U. S. Corps of Engineers, as part of the materials research program in connection with the Kanawha River recanalization project.

The problem was to obtain a mixture that would be sufficiently strong, and at the same time able to take nails, since roofing material was to be nailed to the concrete. Figure 1 shows the results of tests made with different percentages of pine sawdust in a mixture of cement, sand, and sawdust. That proportion of sand was used which would produce a workable mix for the water-cement ratios indicated. It can be seen from the figure that, as the ratio of sawdust is increased, the water-cement ratio and nailability also increase, while the strength and density decrease. Eightpenny nails were used, the nailability of the mixtures being evaluated in terms of the number of days after pouring that each would accept the nails. The concrete was kept continuously wet until the nails were driven.

The mix actually used in roofing the power and operation houses of the Gallipolis Dam was 1:1.75:1.75 of cement, sand, and sawdust, or a ratio of 65 per cent sawdust. It was found in actual practice that eightpenny nails could be driven for approximately three weeks. The compressive strength obtained was slightly greater than 1,000 lb per sq in., and the unit weight about 117.5 lb per cu ft, for a water-cement ratio of sllightly greater than 1.0.

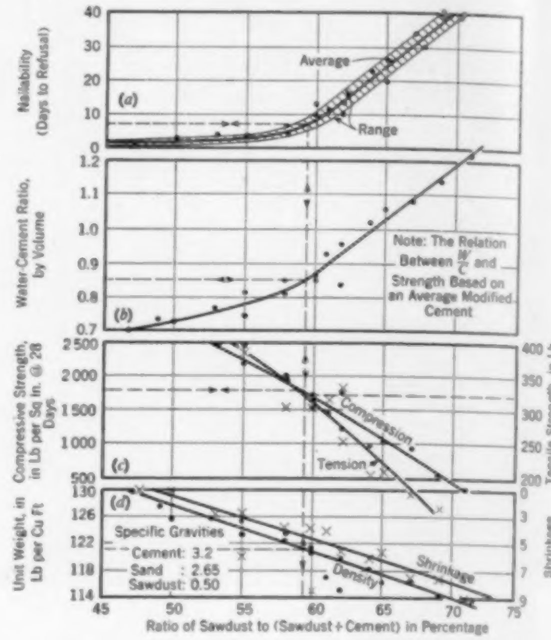


Fig. 1. Relation Between Percentage of Sawdust in Sawdust Concrete and (*a*) Nailability, (*b*) Water-Cement Ratio, (*c*) Strength, (*d*) Shrinkage and Density

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Hydraulic Model Studies for Southern New York Flood Control Project

To the Editor: Major Nold, in the December issue, discusses the hydraulic model studies for the Southern New York Flood Control Project. In this connection additional notes on the construction and operation of the Chenango River model may be of interest.

The 160 by 16-ft model was housed in the locally roofed outdoor canal of the Cornell University laboratory. Cross-sections of the river channel at 50-ft intervals were represented by templates of masonite sheets set 8 in. apart and used as built-in forms for the concrete. Each of the 26 twin-opening piezometers consisted of a broad U of $^3/_{32}$-in. copper tubing with a tee at the mid-point. This U was fastened to the downstream side of one of the (later embedded) 2 by 4's supporting a template on the upstream side. The ends of the U were located at the one-quarter and three-quarters points of the channel width, and were long enough to project above the finished concrete, being pinched shut to prevent plugging during construction. Later the ends were cut off and carefully filed flush with the cement finish. Single copper tubes led from the tees to one of six gage boards, where the upturned end of each copper tube was sealed by a perforated rubber cork into the bottom of a glass tube.

The scales between the glass columns were factory-graduated to 0.5-ft (prototype) intervals on stainless steel tapes.

The small slope—a fall of about 1 in. in the entire model—permitted use of the still-water-level method for the initial setting of the gage scales and for the frequent checking of zeros. Usually, just as in the flow runs, three observers each made a complete round trip, reading all gages twice. Any discrepancy was immediately checked. Auxiliary point and hook gages on cross beams referred the water surface to submerged bench-marks, and also permitted accurate cross-sections of the finished model to be made for checking against prototype surveys.

All the available river surface profiles involved abnormal obstructions in the river bed. These conditions had to be reproduced in the first tests made on the model before a check on river measurements could be expected. The flood of July 1935 threw down the spans of the old Ferry Street bridge, and the wreckage was still in the river during the two floods of March 1936. When a series of accurate profiles was obtained during normal high water early in 1937, the wreckage had been removed but, in the meantime, a contractor's trestle had been placed across the river for the concrete arches of the new Ferry Street (now Clinton Street) bridge. In spite of these difficulties the model results were a good replica of river conditions.

The effects of various suggested local improvements were studied by measurements for three different Chenango River flood volumes, each with three different backwaters representing possible stages in the Susquehanna River. In cases of very small surface fall in a short stretch, the consistency of the data was improved by taking averages of two or three piezometer readings upstream and downstream from the improvements.

Studies of velocity distribution and magnitude across the river were made by motion pictures of groups of 10 to 20 surface and deep floats. Time was indicated by a 12-in. disk, decimally divided, and rotated by a portable phonograph carefully adjusted to one revolution per second. Position was shown by a grid of white

cords stretched across the model at 1-ft intervals. A 16-mm camera pointing vertically downwards was set 9 ft above the water surface. A 15-mm wide angle, f:2.7 lens, at 32 frames per second, was used with three No. 4 photoflood lamps. The paths and velocities of the floats were plotted by projecting the pictures, frame by frame, onto a cross-section sheet, at the proper distance for the desired scale, and plotting the position of each float every quarter of a second.

The construction and experimentation were under the immediate direction of A. N. Vanderlip, Assoc. M. Am. Soc. C.E.

Ernest W. Schoder, M. Am. Soc. C.E.
Professor of Experimental Hydraulics,
Cornell University

Ithaca, N.Y.

Solution of Flat Right Triangles

To the Editor: In the October 1938 issue of Civil Engineering I explained a slide-rule method for finding "The Other Side of a Flat Right Triangle," other than flat triangles being excluded from the scope of the method. In the January 1939 issue James B. Goodwin criticizes this method, but in presenting his argument he chooses a triangle that is far from flat. Further, he fails to insert in the formula his own second trial value of "a."

As set forth in the October issue, x and y are the legs of a flat right triangle, y being the shorter. The hypotenuse, z, is to be found by adding a small difference, a, to x, and the formula is $a = y^2/2x + a$. Mr. Goodwin holds the method responsible for an error of $3^7/_{16}$ in. in the length of the hypotenuse of a triangle in which each leg is 20 ft. However, he finds a value of 8 for a and then fails to use it in the formula. When 8 is used for a in the denominator a better value of 8.33 is found. A careful setting of the slide-rule with the new denominator 48.33, or a near approach to this, gives the final result 8.3 for a. This makes the hypotenuse 28.3, the error being 0.0157 ft, or $^3/_{16}$ in. Of course, this is still far too great a discrepancy when accuracy is desired. However, the method should never be applied to triangles in which the legs are nearly equal, except for rough results. A better example was given in the notes printed in October.

Leonard C. Jordan, M. Am. Soc. C.E.
Consulting and Designing Engineer

New Rochelle, N.Y.

Use of Dunes for Beach Protection

To the Editor: Colonel Hall's valuable paper on the effects of the September 1938 hurricane on the Long Island and adjacent shores, which appeared in the March issue, suggests several subjects of direct interest not only to those engineers concerned with coastal works, but also to those concerned with structures and communications near the coast.

The paper traces the movement of the storm tide wave in the ocean in its reflection from the south coast of Long Island against the north coast of New Jersey and the westward flow of the storm tide in Long Island Sound into the East River. The effects of constrictions and changes in bottom slopes in causing marked local increases in tide heights have been described. These suggest the importance of basing estimates for safe elevations for coastal structures not only on the degree of exposure to storm action, but also on a study of superelevations that may be produced at comparatively sheltered and distant points.

During the inspection of the damage to the shores and beaches, made immediately after the hurricane, it was noted that while the damage to the beaches was great, it was small in comparison to the damage to the structures placed above the level of ordinary high water. In general, the beaches were driven landward, and were eroded to a concave surface back to the line of dunes. The greatest depth of erosion noted was north of Narragansett Pier in Rhode Island, where the foreshore had been eroded to a depth of 4 or 5 ft. In many places the foreshore erosion amounted to only 1 or 2 ft, and at at least one place a beach was raised and widened seaward by the storm. It is probable that the rapid rise of the storm tide, described by Colonel Hall, permitted the waves to break landward of the usual plunge points, and that greater foreshore erosion might have been expected if the beaches had not been inundated.

The protective effect of high sand dunes has been described. The seaward line of dunes left standing after the hurricane usually showed vertical faces on the seaward side, and it was apparent that the erosion would probably have been complete if the hurricane had been of greater duration. Despite their weak resistance to erosion, it should be kept in mind that high and stable dunes are valuable defenses for the lands and structures in the rear, since storms of the hurricane type are generally of short duration. While they may not afford complete protection, they assist and provide time for evacuation of the exposed area.

In contrast to the practice of leveling dunes at building and bathing beach sites reported by Colonel Hall, at critical places in the Netherlands not only is the growth of dunes fostered, but also the dunes are stabilized by planting and protected by barbed-wire fences that limit visitors to certain paths. In this country a notable example of appreciation of the value and scenic worth of stabilized dunes is the work of the Long Island State Parks Commission at Jones Beach.

The comments on the destruction on Fire Island—the barrier beach of Great South Bay and Shinnecock Bay—suggest the need for planning safe routes of evacuation for such areas as cannot economically be protected by seawalls or bulkheads. Had the storm occurred over the Labor Day week-end the loss of life would have been increased many times. Even if there had been time to warn the inhabitants of the threatened areas, the inadequate routes and means of transportation would have prevented the escape of a large part of the population. Zoning of low areas would help, but the effectiveness of this method is reduced by the tendency to build directly on the sands.

Albert C. Lieber, Jr., M. Am. Soc. C.E.
Major, Corps of Engineers, U. S. Army

Washington, D.C.

The Irondequoit Disposal Plant

To the Editor: At the Rochester Meeting of the Sanitary Engineering Division, descriptions of various sewage treatment plants in New York State were presented. Among these was the Irondequoit plant of the city of Rochester, which was also on the schedule of inspection trips. The description, prepared by the writer for that occasion, is presented here for the record.

The west half of the plant was put in operation in 1917. The cost of this part of the plant was \$967,500, including the land, and the addition made to it in 1935 cost \$665,000. Neither of these items included the intercepting sewer, which was constructed at a cost of \$822,200.

In accordance with state requirements, the plant provides for a 45 per cent removal. The cost of treating sewage at this plant is approximately \$3.50 per million gallons, although this estimate varies somewhat from year to year, depending upon the amount of equipment purchased and the repairs necessary.

The sewage treated at this plant has an average pH value of 6.7 to 7.0. The other characteristics are such that a detention period of an hour and three-quarters will produce a removal of 50 per cent of the suspended solids according to Metcalf and Eddy's curve for these tanks. A rate of 120 gal per capita daily has generally been used as a basis for flow calculations. This rate gives a "flowing through factor" of 1.16 cu ft per capita daily. A rate of 4.52 cu ft per capita per year was used as a basis for design for sludge accumulation, and 0.50 sq ft per capita per year for sludge drying beds.

The sewage enters the plant from the interceptor and passes through the coarse racks. Material caught on these racks is removed by hand and disposed of on the screening dump. From the racks the sewage passes through the detritus tanks which slow down the velocity, allowing the inorganic matter to settle out. Material from these tanks is removed by a clamshell bucket and emptied into cars to be used for fill or else removed to the screenings dump. The sewage next passes through the Reinsch Wurl screens, and enters the influent channel leading to the Imhoff tanks. A regulating mechanism gives each tank its proportionate amount of sewage. The capacity of the flowing-through channel on the west side tanks is 185,000 cu ft; the channel on the east side has a capacity of 228,800 cu ft. The sludge settles into the sludge digestion chamber, from which it is removed whenever the beds are

ready to receive it. The sand on the top of the beds has to be replaced from time to time, as some of it is picked up with the sludge in the removal operation. The average number of removals per year is eight. Four days are required to remove sludge from the east beds, and six days for the west beds. After removal from the drying beds the sludge is stored in the sludge dump, where it is available to farmers and golf clubs as a soil conditioner. The effluent passes through the effluent channel to the forebays of the power house, which supplies the plant with electricity, and then through a 66-in. lock bar steel pipe that discharges it into Lake Ontario.

This plant is operated by a crew of about 23 men: a superintendent, who is in charge of all the sewage plants, pump stations, and regulators; 9 men, who operate, clean, and maintain the screens and plant; 12 outside laborers, who draw and remove the sludge and engage in general maintenance and construction work about the plant and grounds; and a truckman and teamster, who perform similar duties when not engaged elsewhere. The latter two spend about 60 per cent of their time at the plant.

This brief description may enable the reader to draw a comparison between plants of modern design and the Imhoff type.

KENNETH J. KNAPP
Sanitary Engineer, City of Rochester

Rochester, N.Y.

Perhaps He Was Not Drinking, After All

DEAR SIR: There is a note of skepticism in your caption to Mr. Ashley's letter concerning snakes, which appeared in your January issue. It implies that the surveyor he quotes may have been indulging too freely in the cup that cheers.

There is, however, no valid reason to doubt the accuracy of the record as reported by Mr. Ashley. In 1905, along Link River near Klamath Falls, Ore., our survey parties witnessed much the same thing. I do not recall that we saw any devouring and disgorging of snakes, but we saw everything else referred to by the early surveyor, and even more—we saw something of the peculiar mating habits of snakes.

JOSEPH JACOBS, M. Am. Soc. C.E.
Consulting Engineer

Seattle, Wash.

The Long Island Hurricane of 1938

TO THE EDITOR: The storm of 1938 on the Long Island and New England coasts has been well described by Colonel Hall in the March issue of CIVIL ENGINEERING. The writer visited a section of the southeastern coast of Rhode Island shortly after the storm and observed its effects there, which were in general about the same as those on the Long Island coast described by Colonel Hall, although modified by topographical differences.

As protective measures against storm waves, Colonel Hall discusses the possible use of sand dunes, bulkheads, and sea walls, and points out that the choice of which device shall be used depends on economic considerations. While sand dunes may be considered a first line of defense, it is one that is easily broken, if not maintained in good condition and at an adequate elevation. Even though the line be of adequate height and dimensions, the necessary breaks in continuity at inlets afford opportunity for water to fill and inundate the ponds in the rear. While direct wave action in the ponds may not be violent, a reversal of the wind blowing the water out may and often does raise the pond level to such an extent as to force the water over the spit, thus eroding it badly or opening new inlets to the sea. This return wave has been known to be more destructive than the direct action of sea waves.

Neither bulkheads nor sea walls will ordinarily be economically justified except in the case of cities of considerable resources. Both are beyond the means of most dwellers on the sea shore.

For those of average means who find it desirable to build on low sandy shores the following suggestions are given:

1. Locate buildings as far back from the water as possible, and always back of the dune line.

2. Do not level off or lower the dunes but, on the contrary, adopt a policy of aiding and promoting their growth by planting suitable beach grasses, and by carefully building up low places in the dune line.

3. Construct buildings on piles driven to a penetration well below sea level, and projecting above it sufficiently to place the main floor above any known inundation level, say, 10 to 12 ft above sea level for the ordinary case. The space on the ground floor may be used as a garage, or for storage.

EARL I. BROWN, M. Am. Soc. C.E.
Colonel, Corps of Engineers, U. S. Army (Retired)

Wilmington, N.C.

Comments on the Maximum Probable Flood

TO THE EDITOR: In the January issue there was an article entitled "The Maximum Probable Flood and Its Relation to Spillway Capacity," by Messrs. Bailey and Schneider. A few thoughts regarding the "maximum probable flood" might not be amiss.

I assume that the authors have in mind the maximum possible flood of all time, although this is not specifically stated. Such a flood is really the maximum probable, as it may occur at any time. It is a real problem to estimate the maximum possible flood from meager rainfall and runoff records extending over a period of about 100 years. In Europe, records of floods on the Seine, Danube, and Tiber rivers, extending over periods of about 300, 900, and 2,000 years, respectively, indicate that even 1,000 years may not be long enough to develop the maximum flood, as on these three streams the greatest floods of record occurred in 1611, 1501, and 1598, respectively. These greatest floods were followed by the second greatest floods at various intervals (272 years on the Tiber and 286 years on the Danube), and German engineers are said to believe that even greater floods can occur on the Danube. These long-time records are entitled to our consideration, although conditions here do not necessarily parallel those in Europe.

In connection with this problem, I would not dismiss use of probability methods as lightly as do the authors. I believe that considerable merit attaches to use of the skew-curve principle, employing probability paper in the manner developed by the late Allen Hazen, M. Am. Soc. C.E., and explained in his book, *Flood Flows*, the object in this case not being to determine 50- or 100-year floods but to estimate the maximum possible flood of all time. For this purpose it should be reasonably safe to assume, say, that the probable 5,000- or 10,000-year flood will be the maximum possible flood.

The merit of this method is indicated to me by the following experience. After the unprecedented Ohio River flood of 1937, I wondered why engineers had been so unprepared for such a flood height, all apparently having assumed that the highest flood of record was the maximum possible flood (this attitude is still too prevalent and should be discouraged). Investigating this question, I ran across Mr. Hazen's book, and prepared probability curves for the Ohio River at Louisville, basing the calculations on principles outlined by Mr. Hazen. The probability curve prepared as of December 31, 1936, indicated plainly that floods 10 to 12 ft higher than the highest flood of record (before 1937) should have been expected at some future time. Substantiating this probability, the 1937 flood came along, reaching a stage about 10 ft higher than the previous record.

A like probability curve, prepared as of December 31, 1938, for the Ohio River at Louisville, indicates that the 1937 flood height will probably be equaled or exceeded about every 280 years on the average (compare with the Tiber and Danube floods previously mentioned), and that still higher floods can be expected to occur. To one who was in the Ohio River valley during the 1937 flood, the prediction of higher floods is not hard to believe.

I agree with the authors that the so-called rational method of calculating storm-water runoff is generally to be preferred to empirical methods. However, the results can be just as incorrect if basic data are lacking or if wrong assumptions are made. The authors have made a welcome contribution toward use of the rational method.

C. FRANK JOHNSON, Assoc. M. Am. Soc. C.E.
Senior Engineer, Commissioners of Sewerage of Louisville

Louisville, Ky.

Social Engineering

To the Editor: I agree with Mr. Doherty, in his article on "The Paradox of Social Progress" in the March issue, that the rapid increase in technological progress is tending to make modern civilization so complex that it is reaching a point where complete breakdown is possible. Yet I believe that more is required than a revamped educational plan such as he suggests for a remedy. I would emphasize instead his appeal for constructive effort. Political freedom in this country is doomed unless for the masses of our people a measure of true economic freedom can be obtained.

True economic freedom does not mean freedom to get rich as fast as the acquisitive instincts of men can devise ways of doing so. It means, on the contrary, a way of organization of economic life which will provide continuous prosperity, abolish poverty, crime, and disease, remove class conflicts, and develop fully the physical, mental, and spiritual capacities of every individual in our society.

There are two kinds of liberty, incompatible with each other. On the one hand, is freedom to amass wealth and power through private ownership or management of industry, trade, and finance; on the other, is the liberty of all wage and salary workers, farmers, and professional classes to have jobs and security of opportunity, to express themselves through organization and political control of government, and to seek a higher standard of well being and culture.

To give real meaning to our aspirations for liberty we must substitute social direction of our economy for the power of private or corporate wealth. This must be done in an American way appropriate to the needs of today and the means of satisfying them. City and regional planning, now being extended to state and national planning, are forms of social direction familiar to engineers. But a much greater extension must be made if anything approaching socialized liberty is to be attained. More power must be given to planning boards, and decisions must be made by carefully selected functional bodies in industry, agriculture, and the professions.

The resulting reduction in or abolition of unemployment would confer even greater freedom on those already employed; choice would be broadened because there would be more jobs. Engineers, industrial managers, and the professional classes generally would enjoy greater freedom in their work if its purpose were to provide the abundant life. Nothing is more obvious in an analysis of present society than the frustration suffered by the engineer as a result of the conflict between technological advance and the restrictions imposed by business uncertainty and widespread poverty.

I urge engineers to read George S. Counts' recent book, *The Prospects of American Democracy*. He shows that our American way is in serious danger from within, not from without, and emphasizes the need for social engineering and for plans whereby we can demonstrate that democracy in an industrial society is workable and efficient.

The essential difference between socialized liberty and the totalitarian state must be made perfectly clear. The one results in the functioning of free workers who know their goals to be abundance and good will to all men, regardless of race, color, or creed; the other is tyranny over all workers by state officials with absolute power, backed by force, and exerted by individuals over whom the workers have no power of removal as in a democracy.

In these days when vital economic decisions are made by government, the men elected to represent the people in our democracy must be greatly superior to those elected in the past. Furthermore, engineers must continue to exert pressure for a Federal Department of Public Works, and for extension of the principle of civil service. Many engineers are now employed in government positions, and many more should enter such service as a life career.

Unless we can succeed in stimulating our productive system and increasing our purchasing power to put the unemployed back to work there can be no permanent answer to our economic dilemma. We must recognize how completely interdependent are all workers and all nations in our modern world. Organizations of employees, employers, and consumers are absolutely essential, and engineers are well qualified to serve as mediators and interpreters between divergent interests. Note the fact that an engineer is serving as administrator of the National Labor Relations Board.

May I again urge engineers to enlarge their field of interest and activity so as to enter into the socially complex issues of our time with greater effort and understanding.

J. P. J. Williams, M. Am. Soc. C.E.
Instructor in Civil Engineering, Cooper Union
New York, N.Y.

Improvements in Highways and the Motor Vehicles

To the Editor: The article by Mr. Upham in the January issue voices the common fallacy that, "The building of highways has failed dismally to keep pace with the output of automobiles" and that "while these changes (improvements) have been taking place in the vehicles, the highways have, in too many cases, remained in their pre-war state."

That highways and highway engineering have made advances fully comparable with the progress of motor vehicle design and construction, can be shown by considering only a few examples from around one city—New York. The Holland and Lincoln tunnels, the George Washington and Triborough bridges and their approaches, the West Side highway, and the Westchester, Long Island, and Merritt parkways, are all just as great advances in highways and highway engineering as are modern cars compared with those of the pre-war period. These highway improvements represent an inversion of public funds of upwards of $500,000,000 in one metropolitan area alone, most of which is a general charge against the taxpayers as a whole.

As a matter of fact, there is hardly a mile of highway in the United States which has not been materially improved since the war. We have developed greatly improved methods of building pavements and road surfaces of all kinds; in short, the highest type of engineering skill is making the taxpayers' dollar give the most service. Our skill in adapting design and methods of construction to the many and greatly varied conditions of our country is quite fairly comparable with the skill of the motor vehicle manufacturers in adapting their vehicles to the services and purses of their users. Generally speaking, our highways easily accommodate the traffic except at times of occasional peak loads and in the vicinity of large centers of population where there is at times severe congestion. The relief of this latter, however, is a problem of finance.

Mr. Upham suggests that this problem could be solved by stopping the diversion of funds contributed by the owners of motor vehicles in the form of gasoline taxes, license fees, and so forth. He estimates that this diversion amounted to $161,413,000 in the United States in 1937. However, if this amount were spent on the 3,120,000 miles of unpaved highways of the country, only $50 per mile additional would be available for improvements, and that would not go very far.

The facts are, as indicated by the experience of the New York-New Jersey metropolitan area, that if we depended only on these fees and taxes from motorists we should not get very far with some of our most pressing highway problems. While there are plausible reasons against the so-called diversion of highway funds, there are others in favor of putting all taxes into the general funds of the government and budgeting all funds for the best interests of all the people. Carried to its logical conclusion, of course, the earmarking of specific taxes for the sole benefit of those who pay them would result in an obvious absurdity, and the motorist would fare much worse than he does now. For the year ending June 30, 1937, the federal government alone spent $337,000,000 for highway construction, is still continuing spending at more or less this same rate, and there are other expenditures from the general funds of states and other governmental units.

The relief of congestion, our most important highway problem today, will require a large amount of money, and this probably cannot be furnished wholly by motorists and motor vehicles. If the money is to come from the general funds, however, criticism of the diversion of motor vehicle taxes is not per se sound, and engineers must consider all projects of public works in terms of the general welfare of the country and not from any narrow, individualistic, or ex-parte point of view.

Mr. Upham states that "one result of this failure of our roads to meet the demands of modern traffic is a steadily increasing death toll in highway accidents." As a matter of fact, nearly all, if not all, careful studies of accidents tend to put most of the blame on the drivers, and on lack of proper maintenance of the vehicles. There is no sound reason why the large percentage of safe, careful drivers should be penalized by excessive and unnecessary expenditures for the faults of a comparatively few reckless and careless individuals. Reform is needed more in the examination of drivers and vehicles than in the construction of highways.

Fred Lavis, M. Am. Soc. C.E.
Consulting Engineer
New York, N.Y.

SOCIETY AFFAIRS

Official and Semi-official

Chattanooga Entertains the Society

1939 Spring Meeting Covers Wide Range of Interests

With characteristic southern hospitality, Chattanooga welcomed the Society for its 1939 Spring Meeting. An unusually large attendance, both at technical sessions and at social functions, reflected the enthusiastic interest of the local members in Society activities and was a compliment to the many from near and far who had had a part in planning the program.

Before the Meeting was under way, vacant rooms at the headquarters hotel were exhausted. And still the visitors poured in. Meeting rooms were crowded to capacity, dining facilities likewise. The final attendance reached over 1,200, which is believed to be a record for any Society Spring Meeting.

The meeting proper opened on Wednesday, April 19, though for the preceding three days the local headquarters had already worn a convention aspect, with meetings of the Board of Direction, of the Local Sections Conference, and of numerous committees almost continuously in progress. At the opening session there was a welcome by the Governor of Tennessee and the Mayor of Chattanooga, and a response by President Sawyer. Then, turning at once to matters of technical interest, the session heard an address on the construction program and activities of the Tennessee Valley Authority.

Following the luncheon recess there was the customary "general meeting," designed to interest the membership at large. Mississippi River flood control, and the manufacture and use of aluminum structural shapes, were the widely different topics that shared this program.

Thursday morning and afternoon and Friday morning were given over to technical sessions, with six of the Technical Divisions taking part. The newly formed Hydraulics Division, for its first appearance on a Society program, sponsored two sessions, with a total of eight papers on experimental hydraulics, field research in hydrology, and other topics. The Surveying and Mapping Division also held two sessions, with attention given to boundary and city surveying, to topographic mapping, and to stereoscopic plotting instruments. A large exhibit of such instruments, on display throughout the meeting, attracted wide attention and interest.

In each of two sessions, the Soil Mechanics and Foundations Division concentrated on the topic of dam foundation treatment, with the case histories of Chickamauga and Guntersville dams as the starting points for discussions. The report of the Committee on Soil Sampling and Testing was also heard.

An extensive symposium on masonry dams, extending over two sessions, was the contribution of the Power Division in cooperation with others. Design assumptions, design of various types of dams, and the investigation and treatment of foundation rock were the topics covered.

Papers on low-cost roads in Alabama and in South Carolina, and on the construction of highway embankments across TVA reservoirs, made up the program of the Highway Division's single session. Three papers were on the Construction Division program—one on time studies in heavy construction, and two on the principles of construction plant layout (for large dams and for buildings).

Adjacent to the meeting rooms a fine exhibit of engineering interests was on display, through the courtesy of the U. S. Geological Survey, the U. S. Engineer Corps, and of course the Tennessee Valley Authority. The organizations also furnished demonstrators who were in continuous attendance. Besides the photogrammetry and mapping instruments, charts, and photographs, there were numerous scale models, including concrete forms, river locks, and movable and stationary dams of several types. Some of the hydraulic working models were a continuous source of interest. A large demonstration of government stream gaging equipment and recorders was also especially notable.

Typical of the enthusiasm was the experience at the Wednesday evening dinner. As reservations multiplied, additional dining rooms were commandeered, until finally the hotel threw up its hands and the lists were closed at about 450 covers. Those attending were well repaid by a fine meal, some excellent negro singing, and a colorful lecture on the National Parks.

Objective of One of the Inspection Trips Was Hiwassee Dam

Another splendid function was the dinner and dance on Thursday evening. The complete facilities of the Chattanooga Golf and Country Club were commandeered for this event. There, in its lovely setting of woods and lawns and fairways, along the curving bank of the Tennessee River across from the city, about 500 merrymakers gathered just before dark. The spacious clubhouse overflowed with the evening's enjoyment. Fine music, a splendid meal, and an artistic floor show were blended with good fellowship and a dance lasting to late hours. In many respects this was the high point of the Meeting, socially.

Other social events, arranged for the ladies alone, were held during the daylight hours of both days, while the technical sessions were in progress. These included motor trips to many points of scenic and historic interest, as well as informal gatherings of smaller groups.

As usual, the student conference deserves more than a word. It ran all day Thursday, including a luncheon. Splendid facilities for a meeting room, provided in a nearby public building, aided in the presentation of a strong program. About 200 were in attendance, mostly students, representing 21 different colleges, scattered from Wisconsin to Louisiana and from Virginia to Texas.

Chickamauga Dam, under construction just a few minutes' drive from Chattanooga, was visited on Friday afternoon by members and their ladies—an inspection trip of extraordinary interest. And to close the Meeting, on Saturday, April 22, there was a choice of four extensive tours to hydroelectric projects of the TVA, the Aluminum Company of America, and the Tennessee Electric Power Company, and to the scenic Great Smoky Mountain National Park.

As a whole the Meeting was a great success. Everyone cooperated 100 per cent. Extended and detailed plans were perfected well in advance. Every committee member, man or woman, took the job seriously. The result was a wonderfully fine Spring Meeting.

Changes Made in Organization of Society's Technical Work

As THE RESULT of a two-year study of the organization of the Technical Department of the Society, the Board of Direction at Chattanooga completed a series of changes in the By-Laws recommended by the Committee on Technical Procedure.

The study indicated that the Technical Divisions have characteristics that may be described as vertical or as horizontal. Vertical Divisions are specific in their fields and in general have phases in common with only one or two other Divisions. Functional Divisions, on the other hand, are general in character and have phases shared with many or all Technical Divisions. Accordingly the following Divisions have been designated as Technical: Highway, Irrigation, Power, Sanitary Engineering, City Planning, Structural, Waterways, Soil Mechanics and Foundations, and Hydraulics. The remaining three Divisions are now referred to as Functional Divisions: Construction, Surveying and Mapping, and Engineering Economics.

Emphasis has been placed on cross representation. This may be attained by means of joint committees and also by the appointment from the Technical Divisions of corresponding members to the Functional Divisions. These corresponding members are attached without vote to the executive committee of the Functional Division.

Another change recommended by the Committee on Research was the allocation of the Society's research committees to appropriate Divisions and the discharge of the Committee on Research, which was the administrative unit. Five research committees having to do with water have been assigned to the Hydraulics Division. The remaining research committee, that on Stresses in Railroad Track, reports at its own request direct to the Committee on Technical Procedure. The Committee on Research has been discharged.

Another change involved the reorganization of the Committee on Technical Procedure. Its personnel now consists of the chairman of each Technical Division, whose term, for the purpose of greater continuity, has been increased from two to three years; two representatives from the Board of Direction, who are to be Vice-Presidents of the Society, each to serve a two-year term; the chairman of the Committee on Publications; and the chairman and secretary of the Committee on Technical Procedure. Formerly the President of the Society was designated as chairman, but in view of the many responsibilities incumbent to the presidency, he has now been released from that function. The new chairman is to be elected for a one-year term, subject to two additional reelections, from among the present or former members of the Committee on Technical Procedure. The Secretary of the Society is the secretary of the Committee on Technical Procedure.

In addition, an Executive Committee of the Committee on Technical Procedure is now provided, consisting of the chairman and four other members elected from the Committee on Technical Procedure. These members serve for their full term. The Executive Committee is responsible for the review of Division reports and budget requests, for continuing studies of efficiency in Division operations, and for the usual interim functions of an executive committee.

The Manual of Procedure for Technical Divisions is being revised with a view to making it more widely available for the information of the Society's membership.

Forecast for May "Proceedings"

Design of an Open-Channel Control Section

By Karl R. Kennison, M. Am. Soc. C.E.

Designing the shape of the controlling section, by a definite mathematical relationship, to produce any desired rating curve.

Tension Tests of Large Riveted Joints

By Raymond E. Davis, Glenn B. Woodruff, and Harmer E. Davis, Members Am. Soc. C.E.

New data observed on modern, high-strength steel specimens.

Flashboard Pins

By Chilton A. Wright and Clifford A. Betts, Members Am. Soc. C.E.

Flashboards in which the supporting pipes fail within narrow limits of reservoir water level.

Miles M. Dawson Next Freeman Scholar

THROUGH the cooperation of the War Department, Miles M. Dawson, Captain, Corps of Engineers, U. S. Army, has been designated as the Society's Freeman Scholar for the year beginning July 1, 1939. Captain Dawson will sail for Europe on May 3, in order to serve as a delegate at the annual meeting of the Permanent International Association of Navigation Congresses in Brussels and as a representative of the United States at the opening of the International Water Technique Exposition at Liége.

ANNUAL CONVENTION of the Society in San Francisco, Calif., July 26–29, 1939

The new appointee, an Associate Member of the Society, has for the past four years been assistant professor of military science and tactics at the State University of Iowa. During this period he has done graduate work in the mechanics and hydraulics department, toward the degree of master of science. He is a graduate of the U. S. Military Academy and holds the degree of C.E. from Cornell University. He has had varied hydraulic experience in the Philippine Islands and as assistant to the District Engineer at St. Louis.

Captain Dawson's project as a Freeman Scholar will be "a study of the technique of European hydraulic design by the use of models, especially a comparative study of the behavior of models and their prototypes." Although most of his time will be spent in Germany, he plans also to visit laboratories and hydraulic works in Switzerland, France, Holland, and Russia. On his return to the United States he will be stationed at Vicksburg, Miss.

New Technical Division Rosters Completed

PREPARATION of new membership rolls for the 12 Technical Divisions of the Society has just been completed. Data for this compilation were supplied by the members on cards distributed from Society Headquarters on December 1, 1938. The new registration in each Division is shown in the following table, which also gives the registrations from the old lists, now discarded.

Division	Registration: New List (March 23, 1939)	Registration: Old List (December 2, 1938)
City Planning	619	1,435
Construction	2,832	2,977
Engineering Economics	781	775
Highway	1,262	2,222
Hydraulics	1,809	71
Irrigation	494	1,068
Power	654	953
Sanitary	1,140	1,765
Soil Mechanics and Foundations	1,391	528
Structural	2,387	3,015
Surveying and Mapping	878	1,200
Waterways	632	1,038
Total	14,879	17,047

It is interesting to note that the newly formed Hydraulics Division is exceeded by only two others in enrollment, and that just below it in registration comes the next newest Division—Soil Mechanics and Foundations.

Comparisons between the old and new lists can best be made by expressing the enrollment in each Division as a percentage of the total enrollment in all Divisions. City Planning, which formerly accounted for 8.4 per cent of that total, has dropped to 4.2 per cent. Construction, though numerically slightly smaller than before, now accounts for 19 per cent of all Division enrollment, as compared with a previous 17.5 per cent. Engineering Economics has also increased slightly, while the Highway Division has dropped from 13.1 to 8.5 per cent of the total.

The Hydraulics Division, negligible in the old list, now accounts for 12.1 per cent of the total. This more than balances the decreases in the Irrigation, Power, Sanitary Engineering, and Waterways Divisions, which together have dropped from 28.3 to 19.6 per cent. There have also been decreases from 17.7 to 16.1 per cent in the Structural Division and from 7.0 to 5.9 per cent in the Surveying and Mapping Division.

Committee on Applied Mechanics

A COMMITTEE on Applied Mechanics has been formed in the Structural Division as the result of a petition with more than 130 signatures which was presented to the Board of Direction in 1938. The Board sought the opinion of the Committee on Technical Procedure as to whether this activity should be carried on jointly by the various Divisions, by a separate Division created for the purpose, or by a special committee under one of the Divisions. The Board approved the recommendation of the Committee on Technical Procedure that a strong Committee on Applied Mechanics be formed under the sponsorship of the Structural Division.

The personnel of the new committee is as follows: E. L. Eriksen (chairman); J. M. Garrelts, H. J. Gilkey, Shortridge Hardesty (Contact Member), E. C. Hartmann, S. C. Hollister, and A. A. Jakkula. Two additional members, R. E. Peterson and S. Timoshenko, have been appointed from outside the Society.

As noted in the 1939 Year Book, the objectives of the Committee on Applied Mechanics are to stimulate and develop studies and research in applied mechanics as it pertains to civil engineering problems, and to promote the coordination of similar work with the Applied Mechanics Division of the American Society of Mechanical Engineers.

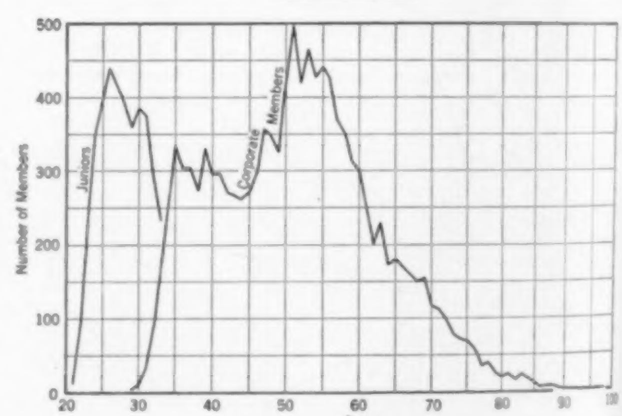
Two World's Fairs on Society Calendar

WITH the forthcoming Annual Convention scheduled for San Francisco and the Fall Meeting for New York, visits to World's Fairs on both coasts can be combined with attendance at Society events in the summer and early fall. Moreover, it is anticipated that special rail rates will make it possible for members to attend the first meeting, return to their homes, and later make the trip to New York, at exceptionally reasonable cost.

The San Francisco Convention, July 26–29, will be of wide technical interest, with nine of the Society's twelve Divisions represented—Soil Mechanics and Foundations, Irrigation, Hydraulics, Waterways, Structural, Power, Sanitary Engineering, Highway, and City Planning. The sessions are so planned that many free hours will be available for visits to Treasure Island, but despite this fact the Society's own social events will retain their customary prominence on the program.

CIVIL ENGINEERING for June will contain a more detailed announcement of the Annual Convention, and the full program will be published in July.

At the Fall Meeting, September 4–9, the Society will be honored by having as its guests the Institution of Civil Engineers of Great Britain and the Engineering Institute of Canada. In addition, the Institution of Mechanical Engineers of Great Britain will be guests during the same week of the American Society of Mechanical Engineers, and a number of events will be planned in common for all these organizations.



MEMBERS FROM 21 TO 98

An interesting study of the age distribution of the members of the Society has just been completed by the Headquarters staff, as part of a study being prepared for the Committee on Salaries. The data are presented in the accompanying chart.

Appointments of Society Representatives

R. L. BERTIN, M. Am. Soc. C.E., has been appointed to represent the Society on the Sectional Committee on Coordination of Dimensions of Building Materials and Equipment of the American Standards Association.

GEORGE W. BURPEE, M. Am. Soc. C.E., has been appointed one of the Society's representatives on the Engineers' Council for

Professional Development to fill the vacancy caused by the death of FRANK E. WINSOR.

THOMAS M. MCCLURE, M. Am. Soc. C.E., has been appointed the Society's representative to the celebration of the fiftieth anniversary of the founding of the University of New Mexico, to be held at Albuquerque, N.Mex., on June 4.

HENRY J. SHERMAN, M. Am. Soc. C.E., served as the Society's delegate at the 43d annual meeting of the American Academy of Political and Social Science, held in Philadelphia, Pa., on March 31 and April 1.

W. E. VEST, M. Am. Soc. C.E., served as the Society's delegate to the centennial celebration of Duke University, Durham, N.C., on April 21, 22, and 23.

News of Local Sections

Scheduled Meetings

ALABAMA SECTION—Meeting at the Whitley Hotel, Montgomery, on May 11, at 7:30 p.m.

ARIZONA SECTION—Spring meeting in Tucson on May 13.

DAYTON SECTION—Luncheon meeting at the Dayton Engineers Club on May 15, at 12:15 p.m.

GEORGIA SECTION—Luncheon meeting at the Atlanta Athletic Club on May 8, at 12:30 p.m.

ILLINOIS SECTION—Luncheon meeting at the Chicago Engineers Club on May 12, at 12:15 p.m.

KANSAS STATE SECTION—Meeting at the Kansan Hotel, Topeka, on May 12, at 6:30 p.m.

LEHIGH VALLEY SECTION—Meeting in Hazelton, Pa., on May 26.

LOS ANGELES SECTION—Dinner meeting at the California Institute of Technology on May 10, at 6:30 p.m.

MARYLAND SECTION—Joint meeting with the Student Chapters at the University of Maryland and Johns Hopkins at the Engineers Club, Baltimore, on May 16, at 8:15 p.m.

METROPOLITAN SECTION—Technical meeting in the Engineering Societies Building, New York City, on May 17, at 8:00 p.m.

MIAMI SECTION—Dinner meeting at the Alcazar Hotel on May 5, at 7:00 p.m. (jointly with Florida State Water Works Association for annual banquet at 8:00 p.m.).

MICHIGAN SECTION—Joint meeting with the Michigan State College Student Chapter at East Lansing, Mich.

MID-SOUTH SECTION—Annual meeting at the Peabody Hotel, Memphis, Tenn., on May 5 and 6. Program will include talks on the coming of TVA power to Memphis and the Mid-South, malaria control, housing problems, the Memphis flood control project, and Sardis Dam. Joint luncheon with Engineers Club of Memphis on Friday; banquet, Friday evening; inspection trip to Sardis Dam, Saturday afternoon.

MOHAWK-HUDSON SECTION—Meeting at Schenectady, N.Y., on May 6 (in conjunction with Student Chapters Conference at Union College).

NORTHWESTERN SECTION—Regular meeting on May 1.

PHILADELPHIA SECTION—Meeting at Engineers Club on May 17, at 7:30 p.m.

SACRAMENTO SECTION—Regular luncheon meetings at the Elks Club every Tuesday at 12:10 p.m.

ST. LOUIS SECTION—Inspection trip to Dam No. 24, Mississippi River, at Clarksville, Mo., the latter part of May. (Get in touch with Secretary Schmickle for details.)

SAN FRANCISCO SECTION—Dinner meeting of the Junior Forum at the London House on May 16, at 6:00 p.m.

SEATTLE SECTION—Dinner meeting at the Engineers Club on May 22, at 6:00 p.m.

TENNESSEE VALLEY SECTION—Dinner meeting of the Chattanooga Sub-Section at the Y.W.C.A., on May 16, at 6:30 p.m.

WISCONSIN SECTION—Dinner meeting at Madison Wis., on May 1, at 6:30 p.m. (jointly with Madison Technical Club).

Recent Activities

ALABAMA SECTION—*Birmingham, March 23:* Substituting for Lee Warren, the scheduled speaker, C. B. Coe, secretary of the Tennessee Valley Section, discussed the forthcoming Spring Meeting of the Society, to be held in Chattanooga. He was followed by Alex O. Taylor, who described the proposed Alabama Technical Society. Mr. Taylor is director of engineering extension and cooperative education at Alabama Polytechnic Institute. The technical program consisted of the reading of a paper entitled "Factional Information Pertaining to Highway Planning," by A. Reese Harvey, Jr., state manager of the State-Wide (Alabama) Highway Planning Survey. Col. A. C. Polk discussed Mr. Harvey's paper briefly.

BUFFALO SECTION—*February 7:* A number of army officers were present at this meeting, the Fourth Annual Engineers Dinner, which honored Maj. Gen. Edward M. Markham and several other prominent members of the Society. Brief talks were given by George S. Minniss, former president of the Section; Arthur W. Harrington, Director of the Society; and Field Secretary Jessup. General Markham, who was introduced by Col. M. C. Kelly, his classmate at West Point, was the speaker of the evening. In his talk General Markham emphasized the fact that waterway transportation is of vital importance to the economic welfare of the nation.

CENTRAL OHIO SECTION—*March 16:* The principal feature of this luncheon meeting was a talk on the WPA secondary road program for Ohio. This was given by Harry Metcalf, chief engineer of the bureau of maintenance of the Ohio State Highway Department. C. V. Youngquist was chosen to represent the Section at the Spring Meeting.

COLORADO SECTION—*Denver, March 13:* Arthur S. Adams was the principal speaker at this session. Dr. Adams, who is professor of mechanics and assistant to the president of the Colorado School of Mines, chose for his topic, "What Is Progress?" In his talk he stressed particularly the rôle of the engineer in present-day life. P. S. Bailey, president of the Section, discussed the activities of American Engineering Council and gave a brief résumé of bills before the state legislature.

CONNECTICUT SECTION—*Hartford, March 15:* On this occasion the Juniors in the Section were hosts to all civil engineers in the state. The program consisted of talks by Francis E. Twiss, assistant in the city engineer's office, and J. H. L. Giles, senior sanitary engineer in the Connecticut State Department of Health. Mr. Twiss gave a talk illustrated with colored motion pictures on the precise mapping program of the metropolitan district, which was started in 1931 and is now well under way. The objectives and methods of modern sewage disposal were discussed by Mr Giles, who showed slides illustrating the high degree of mechanization at modern sewage treatment plants. Both men are Juniors in the Section.

DAYTON SECTION—*March 20:* Joint meeting with the University of Dayton Student Chapter. A write-up of this meeting appears in the "Student Chapter Notes" department of this issue.

DULUTH SECTION—*February 20:* A discussion of the work of the U. S. Weather Bureau as carried out at the Chicago and Duluth stations was the feature of this session, the speaker being F. H. Weck, meteorologist at the Duluth station of the Bureau. Mr. Weck gave the history of the Bureau and discussed the necessity of its forecasts for airplane and other services.

GEORGIA SECTION—*Atlanta, March 13, 17, and 27:* The regular monthly luncheon meeting, on the 13th, attracted a number of students from the Georgia Institute of Technology. The feature of this occasion was a talk on Chickamauga Dam, which was given by Lee G. Warren, project engineer on the construction of the dam. A floor show and musical program enlivened the annual dinner dance of the Section, which took place on the 17th. Field Secretary Jessup was entertained by the board of directors and

other members of the Section at a special luncheon held in his honor on the 27th. Problems of interest to the Local Sections were discussed at this time.

ILLINOIS SECTION—*Chicago, April 7:* The forthcoming Spring Meeting of the Society was discussed by Edward Haupt and W. W. De Berard at this luncheon meeting. The subject of engineering licensing legislation was also discussed, and L. H. Lyle, secretary of the Junior Forum of the Section, gave a résumé of Junior activities.

ITHACA SECTION—*Elmira, N.Y., March 14:* Joint session with the Broome and Steuben Area Chapters of the New York State Society of Professional Engineers. An illustrated lecture on "Design and Construction of the Skyline Highway in Virginia" was the feature of the technical program, the speaker being Harold J. Spelman, district engineer for the U. S. Bureau of Public Roads. A dinner preceded the lecture.

KANSAS STATE SECTION—*Topeka, March 23:* Following a dinner, Alfred Iddles, speaker and guest of honor, was introduced. Mr. Iddles, who is executive assistant to the vice-president of the Babcock and Wilcox Company, spoke on "Problems in Present-Day Boiler Practice." His talk was illustrated with colored motion pictures.

LOS ANGELES SECTION—*March 8:* The program on this occasion was devoted to radio broadcasting. Paul L. Johnson, of the Southern California Telephone Company, gave an account of the problems of radio broadcasting and, by means of a specially installed loud speaker, showed the effect of various frequencies upon broadcasting programs and sound reproduction. Broadcasting from the announcer's point of view as compared with the engineer's was then discussed by M. Frankovich, popular sports announcer.

MARYLAND SECTION—*Baltimore, March 23:* An illustrated talk on "The Use of Geology in Engineering" was given by Mark H. Secrist, associate in geology at Johns Hopkins University. The Section has decided to make an annual award of first-year Junior dues in the Society to the outstanding graduate of each engineering school in its district. Qualities of character and leadership as well as high academic standing will be considered in making these awards.

METROPOLITAN SECTION—*New York City, March 15:* "Manhattan Borough Works" was the subject of discussion at this meeting. The main paper in the symposium was prepared jointly by Walter D. Binger, Commissioner of Borough Works of Manhattan, and Lester C. Hammond, chief engineer of the Borough of Manhattan. The paper covered the planning and design of the East River Drive and Harlem River Drive projects as well as other metropolitan highways in their relationship to the major traffic problems of the city. An enthusiastic discussion followed from the floor. *Junior Branch, March 8* and *22:* On the 8th, Arthur G. Hayden, consulting engineer of New York and pioneer in the development of the rigid frame bridge, presented an illustrated lecture covering the analysis and design of this type of structure. On the 22d, Clarence W. Dunham, assistant engineer with the Port of New York Authority, discussed the practical problems in design and construction of continuous structures.

MICHIGAN SECTION—*Birmingham, Mich., March 14:* Joint meeting with the Oakland County Engineering Society. Following a delicious supper, an informal program of singing and satirical skits was enjoyed. The more technical aspects of the evening consisted of a talk on "The Engineering Work of the State Department of Health." This was given by Edward D. Rich, director of the bureau of engineering of the Michigan Department of Health. Mr. Rich brought out the interesting fact that, whereas before the war the typhoid death rate in certain Michigan communities was 280 per hundred thousand, filtration and chlorination have reduced the rates from this disease to practically zero today.

MOHAWK-HUDSON SECTION—*Albany, March 14:* The principal feature of this meeting was an illustrated address on the mineral resources of New York State. This was presented by D. H. Newland, state geologist of New York State, who called special attention to the influence of mineral resources on the industrial development of the state.

NASHVILLE SECTION—*April 4:* The program at this regular bimonthly meeting consisted of the showing of the Society's lantern slide lecture on the San Francisco-Oakland Bay Bridge. As usual a dinner preceded the meeting.

NORTHWESTERN SECTION—*St. Paul, April 3:* "Water Problems of a Railroad" was the topic of discussion at this session, E. M. Grimes being the speaker. Mr. Grimes is engineer of the water service of the Northern Pacific Railway. An enthusiastic discussion followed his talk.

OREGON SECTION—*Portland, March 17:* A symposium on stream and water pollution in Oregon constituted the program on this occasion. Those taking part in the symposium were Carl E. Green, state sanitary engineer, who discussed the history of stream pollution control in Oregon and the present status of municipal sewage treatment; Fred Merryfield, associate professor of civil engineering at Oregon State College, whose topic was "Problems of Industrial Sewage Treatment in the Willamette Valley"; C. I. Grimm, head engineer of the North Pacific Division of the U. S. Engineer Department, who spoke on the Willamette Valley Project; and R. E. Hickson, principal engineer in the U. S. Engineer Department, who discussed the relationship between water pollution and the maintenance of navigable waterways. Discussion of these papers concluded the evening.

PANAMA SECTION—*Panama City, March 6* and *10:* A number of interesting facts on geophysical prospecting and the equipment used were presented at this gathering by E. R. Shepard. Mr. Shepard developed and patented the microphonic detectors and recording devices which bear his name, and he illustrated his talk by both motion pictures and the display of instruments. On the 10th an inspection trip to watch paving operations on the Albrook Air Field runways was enjoyed. Details of the work were explained by F. H. Lerchen, engineer in charge of the project.

PUERTO RICO SECTION—*San Juan, March 21:* This was the first regular meeting of the year. The program consisted of a talk on harbor development in Puerto Rico, given by Walter J. Truss, engineer in charge of the local office of the U. S. Engineer Office. Gilberto M. Font was elected secretary-treasurer to replace Ernesto A. Soler-Lopez, who has resigned from that office.

SACRAMENTO SECTION—*March 7, 14, 17, 18, 21,* and *28:* The regular monthly luncheon meetings took place on the 7th and every Tuesday thereafter. The speakers who addressed these sessions were B. C. Haynes, head of the meteorology department of the Boeing School of Aeronautics; C. E. Tucker, chief of the State Division of Weights and Measures; Raymond E. Davis, professor of civil engineering at the University of California; and C. T. Mess, in charge of the valuation division of the California Railroad Commission. The third biennial assembly of the San Francisco Section, the Sacramento Section, and the Structural Engineering Association of Northern California convened at Sacramento on March 17. Following a banquet, Edward Hyatt, state engineer, spoke on the history and conception of the Central Valley Project, and Walker R. Young, supervising engineer of the U. S. Bureau of Reclamation, presented a paper describing the design and construction of the project. On the 18th the Section arranged for a field trip to Shasta Dam, now under construction. A number of engineers made the trip.

ST. LOUIS SECTION—*March 27:* Following luncheon and a brief discussion of the Missouri Registration Bill, the speaker was introduced. This was Spencer J. Buchanan, of the U. S. Waterways Experiment Station, who gave an illustrated talk on various phases of soil mechanics.

SAN FRANCISCO SECTION—*February 21* and *March 17:* A number of Student Chapter members from the University of California, Stanford University, and the University of Santa Clara attended the dinner meeting on February 21. The speaker on this occasion was Charles M. Upham, engineer-director of the American Road Builders Association, whose subject was "Engineering Practices in Design and Construction of German Superhighways." On March 17 members of the Section went to Sacramento for the joint meeting described under head of the Sacramento Section. *Junior Forum, March 21:* At this session H. G. Crowle, instructor in civil engineering at the University of California, spoke on California prisons. A critical discussion of the Junior Forum concluded the meeting.

SPOKANE SECTION—*February 10* and *March 10:* Discussion on the subject of the advisability of forming sub-Sections occupied the session on February 10. On March 10 the resignation of

Philip G. Holgren as president of the Section, necessitated by his moving to Seattle, was accepted with regret. H. E. Phelps was elected president in his place, and E. L. Haines was elected first vice-president. Mr. Haines will also continue to serve as secretary-treasurer, and he will be assisted by W. S. Mortimer, who has been appointed assistant secretary. Fred M. Brown has been appointed the Section's Contact Member for the Montana State College Student Chapter.

Syracuse Section—*March 13:* A. J. Yeats, structural engineer for the Portland Cement Association, addressed a large gathering on the subject of architectural uses of concrete, illustrating his talk with lantern slides. Preceding the lecture there was a dinner, at which certificates of life membership were presented to R. E. Danforth and H. N. Cole. The latter then described some of his engineering experiences.

Tacoma Section—*March 14:* A complete picture of the water resources investigations of the U. S. Geological Survey was presented at this gathering. The speakers in the symposium were Glenn L. Parker, Arthur Johnson, George M. Thayer, Irving E. Anderson, and A. M. Piper, all of whom are connected with the Survey in some capacity. The discussion was led by J. B. Fink and C. J. Bartholet. Samples of maps and charts and instruments and pieces of field and office equipment, which were on display, added to the interest in the subject.

Tennessee Valley Section—*Chattanooga Sub-Section, December 20, January 17, February 21,* and *March 8:* On December 8, P. H. Wood, chairman of the Chattanooga Housing Authority, gave a talk on the Chattanooga Housing Project. There was a dinner on January 17, and on February 21 the members met to see an interesting motion picture on the construction of Chickamauga Dam. This was shown by J. B. Hays, construction engineer on the project. The first of a series of weekly luncheon meetings, to be held each Wednesday, took place on March 8. *Muscle Shoals Sub-Section, January 12:* Joint meeting with the American Chemical Society and the American Institute of Electrical Engineers.

Toledo Section—*March 30:* Following a dinner, George N. Schoonmaker, director of public service for the city of Toledo, presented a paper on "Modern Water Works Practice" and then described Toledo's public water supply. Many took part in the discussion that followed.

Utah Section—*Salt Lake City, March 24:* A general discussion on the subject of state planning occupied this session. The principal speaker on the program was Sumner Margetts, director of the Utah State Planning Board, who presented the different aspects of the subject.

Wyoming Section—*Cheyenne, March 24:* Joint meeting with the Cheyenne Engineers Club. The program presented on this occasion was entirely in the hands of Juniors, and two of them—Walter W. Flora and Tolliff R. Hance—presented papers. Mr. Flora, who has served as project engineer on several rural electrification projects, had prepared a paper on rural electrification work in Wyoming to date, while Mr. Hance's subject was "A Junior Member Looks at Unionism." A lively discussion concluded the meeting. The Section has decided to make two annual awards consisting of the initiation fee into the Society to civil engineering seniors at the University of Wyoming.

Student Chapter Notes

Bucknell University—*February 22, March 15* and *27:* The accompanying photograph shows some of the members and guests of the Bucknell University Student Chapter who enjoyed a supper meeting held at the university's recreation center at Cowan, Pa.,

Members and Guests of Bucknell University Student Chapter Who Met on February 22 at Supper Party

on February 22. In the afternoon Wayne Yarnall and Charles F. Millard showed slides of Norris Dam. Then came supper and an informal discussion period, during which Field Secretary Jessup talked about Society affairs. On March 15 the members enjoyed a motion picture entitled "Pipe and the Public Welfare," and on the 27th there was a joint meeting with the Pennsylvania State College Student Chapter. On the latter occasion the program consisted of the presentation of papers by the following students: Karl Mason, Curt Yamas, and Donald Shaffer, of Pennsylvania State College; and Charles Eyer and Donald Drumm, of Bucknell University.

Pennsylvania State College: Members of this Chapter recently enjoyed two interesting meetings. The program for one of these was furnished by the Titan Metal Manufacturing Company, of Bellefonte, Pa., who showed two technicolor films illustrating the manufacture of brass welding rods and forgings. C. E. Myers, consulting engineer of Philadelphia and chairman of the Board for the Registration of Professional Engineers in Pennsylvania, addressed the other meeting on "Registration of Engineers in Pennsylvania." The wide interest in this subject was attested by the largest attendance of the year.

University of Dayton—*March 20:* Members of the Dayton Section and other engineers were guests of the Student Chapter for a dinner meeting. The after-dinner speeches were given by the Rev. Dr. John A. Elbert, president of the university; Frank J. Lasar and William Hill, respectively president and secretary of the Student Chapter; and R. K. W. Tom and Michael Sullivan, members of the Chapter. J. E. Root, Director of the Society, was also present and gave a short talk on the advantages of Junior membership in the Society to an engineering graduate. An inspection tour of the engineering laboratories and some spirited games concluded the evening. On another occasion members of the Chapter enjoyed a three-reel motion picture on brick and its usage in road construction, which was shown by H. M. L. Hendrikx, of the Ohio Paving Brick Manufacturers Association.

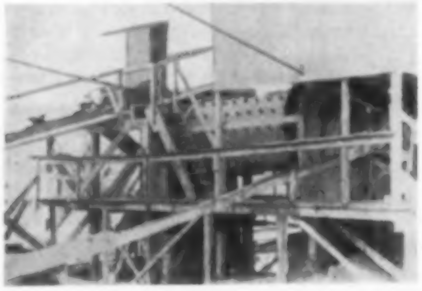
Members of the University of Dayton Student Chapter

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for June

One of the factors markedly affecting the cost of a concrete dam is the availability of suitable aggregate. How such material is produced today, in large quantities and at low cost, is told in an article by M. P. Anderson, scheduled for the June issue. "The trend in modern aggregate production plants," he writes,



U. S. Bureau of Reclamation

Rod Mill Installation, Marshall Ford Dam Aggregate Plant

"is toward better machinery the extensive use of belt conveyors, the interlocking of electrical controls, and the structural use of concrete and steel. . . . Especially has progress been made in the processing of the fine aggregate. Experience on at least one large dam indicates that good concrete sand can be made from suitable rock if the proper machine is used for the crushing process."

"A Modern Building Code—Arrangement and Organization" is the title of an article by D. S. Laidlaw which contains many valuable suggestions for those concerned with the drafting of such documents. Mr. Laidlaw does not discuss technical provisions, but concentrates rather on sequence, indexing, wording, and other factors that must be given attention to make the code of maximum value to its users.

Most of the other papers in the June issue will be selected from those presented at the Chattanooga Meeting. There will be at least one on hydraulic research, one on dam design, one on highway construction, and one on flood control.

International Cooperation on Standardizing Symbols

The American Standards Association is participating in a plan for cooperation with the British Standards Institution, in the standardization of the letter symbols of the mathematical equations used in science and technology. The plan is that before symbols standards are issued independently in both nations, correspondence shall be conducted so that the documents as finally issued will agree as nearly as possible.

At the present time, even though people in the United States, and in Canada, England, and other parts of the British Empire, speak nearly the same language, there is great diversity in the letter symbols used in textbooks and other scientific publications. This is in spite of the fact that books published in each nation are used freely in the others, that colleges exchange students, and that engineering concerns in each country have affiliates overseas. Hence, standardization in each nation, without the cooperation which is being arranged, would be comparatively futile.

The action of the A.S.A. is in response to a suggestion of its own Sectional Committee on Letter Symbols. The chairman of this committee is Dr. J. Franklin Meyer of the National Bureau of Standards at Washington, D.C., and the vice-chairman is Dr. Sanford A. Moss, formerly engineer of the Supercharger Department of the General Electric Company at Lynn, Mass. Dr. Moss, on a recent visit to London, sounded out various English scientists and officers of the British Standards Institution, and found them receptive to the cooperative proposal.

Dnieprostroy Dam

On the page of special interest, or frontispiece, of this issue is shown a view of the Dnieprostroy Dam—a part of the Dnieper River power and navigation project, U.S.S.R.—passing a flood of 500,000 cu ft per sec over its spillways during construction in 1932. A peak flood of 835,000 cu ft per sec occurred in May 1931. The spillway, which is designed to pass 1,250,000 cu ft per sec, has 47 crest gates, each 43 ft wide and 32 ft deep.

The dam was designed and supervised by the late Hugh L. Cooper, M. Am. Soc. C.E., for the Soviet Government, and was built 1927–1932. The following facts concerning it give a measure of its size and capacity:

Concrete content, total	1,600,000	cu yd
Over-all length of masonry structure	5,000	ft
Length of spillway section	2,000	ft
Maximum height of dam above rock foundation	200	ft
Average operating head	116.5	ft
Number of generating units	9 @ 62,000	kw
Average annual power production	2,520,000,000	kwhr

There are triple-flight navigation locks, each chamber of which is 60 ft wide and 450 ft long. The total lift is 123 ft and the draft is 12 ft. The total cost of the project, including the navigation works, was $110,000,000.

International Problems Discussed at Meeting of Social Science Academy

The opening session at the forty-third annual meeting of the American Academy of Political and Social Science, held in Philadelphia on March 31 and April 1 was devoted largely to foreign efforts to win over Latin America. The Society's representative at the meeting, Henry J. Sherman, M. Am. Soc. C.E., has supplied an interesting account of the discussions, from which the following excerpts have been taken:

At this session, he writes, it was brought out that "Foreigners in general, and Germans in particular, have great influence. As Germany has mapped a future Europe so has she mapped South America, covering the Argentine, Brazil, and Uruguay with no boundaries between countries. . . .

"The results of the Lima declaration, it was asserted, have received too little attention as this was the most important Latin American conference ever held, and a strong document came out of it which, delegates felt, represented the ideals of the respective countries. It was, in truth, a Declaration of Independence of twenty-one countries toward totalitarian states.

"'Democracies on the Defensive' was the subject for one session. The importance of mineral resources was developed by Prof. C. K. Leith, of the University of Wisconsin. He pointed out the vital necessity of these resources, which are largely owned by the democracies . . . . Germany, Italy, and Japan . . . have some low-grade ores and oil but far less than their needs. . . .

"Speakers on 'Social Responsibilities of Business' were quite alive to changed conditions in business, developing during the past forty years and now reaching a swift culmination under a new order. The engineer was praised for his contribution to the advancement of industry, and it was stated that his exact thinking should be utilized in the field of economic development and long-range planning.

"In the talk on 'Trading with Dictators' it was brought out that the present United States policy of equality of treatment and playing no favorites was resulting in a normal trade expansion. . . .

"Clarence Streit, author of a book on the Atlantic Union Plan, made a plea for a union of democracy as a nucleus of world government on the lines of our forty-eight states. This is difficult, of course, but not as much so as in 1787. The time has come, he stressed, when we must think along new lines in our relations to Britain, France, and other democracies, if we are to preserve civilization and prevent chaos."

American Road Builders' Association Elects New Officers

At its annual convention, held in San Francisco from March 6 to 11, 1939, the American Road Builders' Association reelected the following officers to serve during the coming year: Murray D. Van Wagoner, president; and Paul B. Reinhold, E. D. Kenna, Lion Gardiner, and Stanley Abel, vice-presidents. Mr. Van Wagoner is a member of the Society, as is George F. Schlesinger, who was elected treasurer of the Association. The list of directors elected for the term ending in 1942 includes several members of the Society—C. E. Myers, W. A. Young, T. H. Cutler, W. A. Van Duzer, and Victor J. Brown.

Brief Notes from Here and There

American Standards Association's new annual list of American Standards and Safety Codes includes some 400 entries in a wide variety of fields. Many of them relate to products, materials, or processes of interest to the civil engineer. In each case, the standard represents general agreement on the part of maker, seller, and user groups as to the best current practice in the industry or process covered. Copies of the index can be obtained free of charge from the American Standards Association, 29 West 39th Street, New York, N.Y.

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A project for the computation of mathematical tables, sponsored by Dr. Lyman J. Briggs, Director of the National Bureau of Standards, is being conducted by the WPA for the City of New York. Tables already computed or now in progress include among others: sines and cosines for the range from zero to 25 radians at intervals of 10^{-3}, to 8 places of decimals; the first 10 powers of the integers from 1 to 1,000; and natural logarithms of integers from zero to 100,000 to 16 places of decimals. Now under consideration, among others, are certain geophysical and astrophysical tables. The directors of the project are anxious to receive suggestions for other compilations. These should be addressed to Dr. Arnold N. Lowan, Chief Project Supervisor, Mathematical Tables Project, 475 Tenth Avenue, New York, N.Y.

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To publicize the latest results of technical research is one of the aims of the "Second Annual Research Week," to be celebrated in the New England States May 15–20. The activities are sponsored by the New England Council and the Engineering Societies of New England, Inc., with other organizations cooperating. Regional meetings and exhibits are to be held in Springfield, Worcester, and Boston, Mass.; Burlington, Vt.; Durham, N.H.; Augusta, Me.; Providence, R.I.; and Hartford, Conn.

* * * *

Stevens Institute of Technology announces its ninth annual economics conference of engineers, to be held at Johnsonburg, N.J., June 24 to July 3. This year the American Society of Mechanical Engineers and the American Institute of Mining and Metallurgical Engineers are joint sponsors. Six sessions daily are scheduled for nine days, dealing with the United States patent system, labor and wages, taxation for business control, industrial psychology, job evaluation and merit rating, and cost analysis and control. As usual, a number of prominent industrialists, economists, and professors have been selected to address the evening meetings.

* * * *

Over 2,000 technical publications from all parts of the world are regularly received by the Engineering Societies Library and reviewed by the Engineering Index Service. Two-thirds of them are in English, but more than half are published outside the United States. Annotated index cards are prepared for all important articles and classified into 281 subject divisions, so that each subscriber to the service can receive, weekly, cards in the subjects of specific interest to him. Details can be obtained from Engineering Index Service, 29 West 39th Street, New York, N.Y.

NEWS OF ENGINEERS

Personal Items About Society Members

Harvey O. Schermerhorn has been appointed commissioner of highways of New York State, succeeding Arthur W. Brandt. Mr. Schermerhorn has been in the service of the state for thirty-five years—for the past four as commissioner of canals and waterways. His headquarters will continue to be in Albany.

Orval J. Baldwin is now senior hydraulic engineer for the National Resources Committee in Washington, D.C. He was formerly assistant professor of civil engineering at the University of Iowa and, also, planning engineer for the Iowa State Planning Board.

Andrew J. Little, Jr., since 1933 field engineer for the PWA in Florida, has been appointed assistant state administrator for WPA in Florida.

Albert A. Brensley has resigned as sanitary engineer and superintendent of the Department of Sewerage and Sewage Treatment of the City of Kankakee (Ill.) in order to establish a consulting engineering practice there. He will specialize in sewage treatment works and sewage.

W. F. Way and A. S. McLean have formed a partnership to take over the contracting practice of Stuart Cameron and Company, Ltd., of Vancouver, B.C. Mr. Way was formerly construction superintendent for this company.

Elwood T. Nettleton, for the past fourteen years western sales manager and research engineer for the New Haven Trap Rock Company, has resigned to become engineering director and secretary of the New York State Crushed Stone Association. His headquarters are in Albany, N.Y.

H. R. Helland, consulting engineer of San Antonio, and W. O. Washington, county engineer of Cameron County, have been named planning engineers for the Texas State Highway Department, with headquarters at Laredo, Tex.

Edward N. Todd, former state highway engineer of Kentucky, has been assigned as resident engineer in the field office of the Municipal Housing Commission on Project Ky-1-1, Louisville, Ky., a low-rent housing project authorized by the U. S. Housing Authority.

Albert H. Hinkle, until recently director of the Kentucky Rock Asphalt Institute at Louisville, Ky., has accepted a position with the Asphalt Institute as district engineer with offices in Cincinnati, Ohio.

Steven Malevich is now employed as a junior bridge designer with the Pennsylvania Turnpike Commission, with headquarters at Harrisburg, Pa. He was previously a draftsman for the American Bridge Company at Ambridge, Pa.

T. W. Brannan, for a number of years assistant engineer for the Ohio State Highway Testing Laboratory, has accepted the position of engineer for the asphalt sales department of the Standard Oil Company (Ohio), with headquarters in Columbus.

Raymond Archibald, who for the past three years has been connected with the U. S. Bureau of Public Roads in the capacity of bridge engineer on inter-American highway work at San Jose, Costa Rica, has joined the staff of the J. E. Greiner Company, consulting engineers of Baltimore, for whom he will serve as resident engineer on the construction of a bridge over the Potomac near Newburg, Md.

S. W. Jackson is now district engineer for the Pennsylvania State Highway Department, with headquarters at Franklin, Pa.

Gordon M. Fair, professor of sanitary engineering at the Harvard University graduate school of engineering, has been elected president of the Boston Society of Civil Engineers.

W. R. Hutchins has been made state highway engineer of Arizona. Mr. Hutchins has been connected with the Arizona State Highway Department since 1923, his most recent post being that of district engineer.

James S. Naismith is now assistant engineer for Myers and Noyes at Dallas, Tex.

Edward J. Kelly, mayor of Chicago for the past six years, has just been reelected for a four-year term. Mr. Kelly has been in the service of the city of Chi-

cago since 1894 when he joined the Sanitary District of Chicago as an axman. He was promoted many times, finally becoming chief engineer. On several occasions he was loaned by the Sanitary District to the State of Illinois, and he served as president of the Board of South Park Commissioners from 1924 to 1933. In the latter year he

EDWARD J. KELLY

was appointed mayor of Chicago to fill the unexpired term of the late A. J. Cermak, and in 1935 he was elected to this office by a record vote.

ELLWOOD D. POWERS, consulting engineer of Newark, N.J., announces the formation of a partnership with Richard P. Dee to be known as Powers and Dee, consulting engineers. The new firm will continue the business conducted by Mr. Powers from 1920 to 1934 under the name of Eugene S. Powers and Son, Inc., and for the past five years under his own name.

ANTHONY P. DEAN, formerly connected with the Division of Operations of the U. S. Forest Service, was recently appointed assistant regional forester in charge of the Division of Engineering of the California Region of the Service. His headquarters are in San Francisco.

WEBSTER L. BENHAM has been made manager of the Oklahoma Concrete Pipe Association. Previously Mr. Benham was director of operations and chief engineer for the WPA in Oklahoma.

CLARENCE E. BARDSLEY, until recently professor of hydraulic engineering at the University of Missouri, has joined the staff of the Oklahoma Agricultural and Mechanical College in the capacity of professor of civil engineering in charge of courses in hydraulic engineering.

D. A. WEIR is now field engineer for Doyle and Russell, contractors of Richmond, Va., on the construction of school buildings in Morgantown and Monongahela counties, West Virginia.

ROLAND VOKAC has been elected a member of Sigma Xi. The initiation ceremony took place on April 26, 1939, at the University of Michigan, his alma mater (1929).

CHARLES E. CONOVER, who for the past thirty-eight years has been associated with the work of rapid transit construction in New York City, has retired on account of ill health and is living at his home, 274 South Middletown Road, Pearl River, N.Y. For the past nine years Mr. Conover has been the division engineer in charge of the Division of Designs with the Board of Transportation.

FREDERICK H. WEED has resigned as division engineer of the Flood Control Bureau of the Water and Power Resources Board of the State of Pennsylvania to become connected with the Federal Power Commission. Mr. Weed has been assigned to the same project on which he has been working for the State of Pennsylvania—that is, the flood control reservoirs above Pittsburgh.

KENNETH L. DE BLOIS is now a structural engineer in the Corps of Engineers, U. S. Army, with headquarters in Pittsburgh, Pa. He was formerly associate highway bridge engineer in the U. S Bureau of Public Roads at San Francisco.

ISIDORE DELSON, formerly chief of the bridge design section of the Bureau of Bridges, New York City Department of Public Works, has been appointed assistant to HOMER R. SEELY, deputy commissioner in charge of bridges.

LEONARD J. SAMPPALA has gone to Battle Creek, Mich., where he will represent Shoecraft, Drury and McNamee, of Detroit. He was formerly regional engineer for the WPA at Lansing, Mich.

DECEASED

WILLIAM TILLOTSON GOULD (M. '90) of Burke, Va., died at Hastings-on-Hudson, N.Y., on February 11, 1939, at the age of 80. Mr. Gould's early career was spent in railroad work—much of it with the Northern Pacific. From 1904 to 1911 he was assistant engineer for the Pennsylvania Railroad; from 1911 to 1913 expert aide in the U. S. Navy Department; and from 1914 to 1917 chief engineer and director of the Nevada–California–Oregon Railway. In 1919—after a period overseas as captain in the Corps of Engineers, U. S. Army—he resumed his position with the latter railway and also served as chief engineer of the Southern Pacific Company. He retired in 1930.

FREDERICK STUART GREENE (M. '12) of Albany, N.Y., died suddenly in Washington, D.C., on March 26, 1939, at the age of 68. On account of failing health Colonel Greene recently resigned as superintendent of public works of New York State after twenty years in the service of the state. From 1890 to 1917 he was actively engaged in engineering work, and in 1919 he became state superintendent of highways. Later he served as superintendent of public works during several administrations. During the war he went overseas with the Combat Engineer Regiment, 77th Division, as a battalion commander. Colonel Greene was also the author of short stories and magazine articles.

MORTIMER JOHNSON MCCHESNEY (Assoc. M. '18) area engineer for the WPA at Beckley, W.Va., died on January 3, 1939, at the age of 53. In 1911, after early experience in Pittsburgh and Charleston, W.Va., Mr. McChesney became assistant city engineer of Charleston, remaining in that capacity until 1917. Later he was city engineer and city manager. At one time Mr. McChesney also maintained a consulting practice in Charleston, and for some years he was engineer for the Board–Haley Company of that city. He became connected with the WPA in 1936.

The Society welcomes additional biographical material to supplement these brief notes and to be available for use in the official memoirs for "Transactions."

HERMAN SCHNEIDER (Assoc. M. '02) dean of the college of engineering at the University of Cincinnati, died on March 28, 1939, at the age of 67. Dr. Schneider was an instructor in civil engineering at Lehigh University from 1899 to 1903, going in the latter year to the University of Cincinnati. There he put into practice the cooperative system of education, whereby academic theory and industrial practice are coordinated, of which he was founder. He was professor of civil engineering at the University of Cincinnati from 1903 to 1929, and president of the university from the latter year until 1932. During the war Dr. Schneider served as chief of the industrial service section of the U. S. War Department.

FREDERIC JACKSON TAYLOR (M. '19) of Livingston, Mont., died about three years ago at the age of 73. Mr. Taylor spent his entire career with the Northern Pacific Railway, starting as transitman in 1886. From 1892 to 1903 he served as assistant engineer at various places in Montana, Idaho, and Washington, and from the latter year on he was division engineer for the district between Mandan, N.Dak., and Paradise, Mont. In 1919 his title was changed to that of district engineer, but he continued in the same position until his retirement in 1933.

RALPH MERVINE WARFIELD (M. '21) in charge of the U. S. Navy public works division of the 3d Naval District, was stricken with a fatal heart attack in New York City on March 21, 1939, while en route to his home in Montclair, N.J. He was 58. From 1910 to 1915 Admiral Warfield was principal assistant at the Puget Sound Navy Yard, and during the war he served as public works officer of the Pensacola Naval Air Station, with the rank of lieutenant commander. From 1917 to 1922 he was aide to the military governor of Santo Domingo, and later he served in the Portsmouth (N.H.) Navy Yard and the Naval District headquarters in Panama and Nicaragua. He was made a captain in 1935, and three months ago

he was promoted to the rank of rear admiral and made public works officer of the 3d Naval District.

George Leverett Wilson (M. '90) engineer, maintenance of way, Twin City Rapid Transit Company, Minneapolis, Minn., died on April 1, 1939, at the age of 84. In 1884, after several years on railroad surveys and construction work in the West, Mr. Wilson joined the staff of the Water Department of St. Paul (Minn.). From 1885 to 1903 he was assistant city engineer of St. Paul, and from the latter year until his death engineer of maintenance of way for the Twin City Rapid Transit Company, in charge of the construction and maintenance of all tracks and roadways on the system.

Changes in Membership Grades

Additions, Transfers, Reinstatements, and Resignations

From March 10 to April 9, 1939, Inclusive

Additions to Membership

Alexander, Verne (Assoc. M. '39), Res. Engr., U. S. Geological Survey, Capitol Bldg., Oklahoma City, Okla.

Bailey, William Jackson (Assoc. M. '39), Office Engr., TVA, Chickamauga Dam, Chattanooga, Tenn.

Blair, Tom Arthur (M. '39), Chf. Engr., A. T. & S. F. Ry., Western Lines, Santa Fé Bldg., Amarillo, Tex.

Brooks, Walter Thomas (Assoc. M. '39), Area Engr., 3d Area, WPA (Res., 2706 Magnolia Ave.), Knoxville, Tenn.

Chollar, Allan Lee (Assoc. M. '39), Res. Engr., State Highway Dept., Box 186, Gatesville, Tex.

Cotton, James Arnold (Jun. '39), Asst. Engr., U. S. Engr. Dept., U. S. Engr. Office, Galveston, Tex.

Davis, Philip (Jun. '39), With Design Section, U. S. Engr. Office, Mobile, Ala.

Davis, William (Jun. '39), 1002 Lowry St., Duquesne, Pa.

Dike, Charles Anthony (Jun. '38), Junior Civ. Engr., U. S. Bureau of Reclamation, Coulee Dam, Wash.

Dodds, John Brydone (Jun. '38), Surveyman, U. S. Engr. Dept., 754 Central Bldg., Seattle, Wash.

Forman, Fred Paton (Jun. '38), 471 Heights Rd., Ridgewood, N.J.

Frisius, Edward Nathaniel (Jun. '39), Care, Socony Vacuum Co., 62 Sharia Ibrahim Pasha, Cairo, Egypt.

Galbraith, Richard Hugh Courtney (Assoc. M. '38), 5 Michie St., Roslyn, Dunedin, New Zealand.

Genthon, Vincent Paul (Jun. '38), 396 Clifton Ave., Clifton, N.J.

Ginsberg, Samuel (Jun. '38), 258 West 97th St., New York, N.Y.

Good, Edward Albin (Jun. '38), 401 South Harding St., Indianapolis, Ind.

Grimm, Nelwin Cleatis (Jun. '38), Tolono, Ill.

Guerin, Frederic William (Assoc. M. '39), Senior Civ. Engr., State Dept. of Public Works, 100 Nashua St., Boston (Res., 7 Harvard St., Worcester), Mass.

Hessler, Ulrich Schneider (Jun. '39), Sub-Insp., U. S. Engrs., Memphis, Tenn.

Johnson, John Walter, Jr. (Jun. '38), Box 527, Blacksburg, Va.

Kyler, James William (Jun. '38), 3342 Woodburn Ave., Cincinnati, Ohio.

Ludasy, Marcell (Assoc. M. '39), Bridge Designer, State Highway Dept., Room 316, State House Annex, Trenton, N.J.

Marshall, Donald George (Jun. '39), 4251 West Irving Park Boulevard, Chicago, Ill.

Parkinson, Clyde Philip (Jun. '39), Junior Hydr. Engr., U. S. Geological Survey, 526 Federal Bldg., Albany, N.Y.

Petersen, John Junior (Jun. '39), 52½ East Daniel St., Champaign, Ill.

Prokopp, Edward Joseph (Jun. '38), Estimator, J. L. Peters Co., 15374 Lauder Ave. (Res., 6004 Iroquois Ave.), Detroit, Mich.

Quattlebaum, Alexander McQueen (Jun. '39), Asst. Prof., Civ. Eng., Clemson Coll., Clemson, S.C.

Raffin, Bennett Lyon (Jun. '39), 54 Palm Ave., San Francisco, Calif.

Reynolds, Don Potter (Jun. '39), Junior Eng. Aide, Dept. of Public Service, City of Toledo, Safety Bldg. (Res., 718 Brighton Ave.), Toledo, Ohio.

Ritz, Francis Benjamin (Jun. '38), 23 Henchman St., Worcester, Mass.

Robbins, Harold Moyer (Jun. '39), Draftsman, Herrick Iron Works, 17th and Campbell St., Oakland (Res., 370 Staples Ave., San Francisco), Calif.

Rose, William Allen (Assoc. M. '39), Asst. Prof., Structural Eng., New York Univ., University Heights, New York, N.Y.

Schmitt, Charles Norman (Jun. '39), Structural Engr., The Proctor & Gamble Co., Ivorydale (Res., 214 Wentworth Ave., Wyoming), Ohio.

Scoralick, Henry Walter (Jun. '39), San. Insp., Westchester County Dept. of Health, County Office Bldg., White Plains (Res., 952 Pelhamdale Ave., Pelham Manor), N.Y.

Shaw, Barton Harrington (Jun. '39), Junior Engr., U. S. Engr. Office, Federal Bldg. (Res., 2107 Duck Creek Rd.), Cincinnati, Ohio.

Smith, George William (Jun. '39), Junior Bridge Engr., State Bridge Dept., Box 1499, Sacramento, Calif.

Smith, John Rockwell (Jun. '39), Junior Engr., City of Henderson, City Bldg. (Res., 226 South Elm St.), Henderson, Ky.

Souder, Byron Norvin (Jun. '39), 8 Sumner Rd., Cambridge, Mass.

Stocking, Frank McKee (Assoc. M. '39), Chf. Engr., State Dept. of Public Lands, Public Lands—Social Security Bldg. (Res., 321 Nineteenth Ave., East), Olympia, Wash.

Taffel, Frank (Jun. '38), 1507 Popham Ave., New York, N.Y.

Taylor, Robert Louis (Jun. '39), Care, U. S. Geological Survey, Post Office Bldg., Ocala, Fla.

Taylor, Seymour Stillman (Jun. '39), Designing Engr., Booth-Thompson Div., Galigher Co. (Res., 3800 South Mill Creek Rd.), Salt Lake City, Utah.

Teuscher, Ivan Maxwell (Jun. '39), With Bureau of Reclamation, 2560 Orchard Ave., Ogden, Utah.

Thornbury, Harry Thomas, Jr. (Jun. '39), Engr., Roberts Nash Constr. Corporation, 39-15 Main St., Flushing (Res., 2608 Ditmars Ave., Astoria), N.Y.

TOTAL MEMBERSHIP AS OF APRIL 9, 1939	
Members	5,639
Associate Members	6,346
Corporate Members	11,985
Honorary Members	27
Juniors	3,971
Affiliates	74
Fellows	1
Total	16,058

Turnburke, Vernon Palmer (Affiliate '39), Gen. Auditor, G. N. Ry., 825 Great Northern Bldg., St. Paul, Minn.

Walther, Carl Hugo (Jun. '39), Checker, Bethlehem Steel Co., Keim St. (Res., 517 High St.), Pottstown, Pa.

Williamson, James Stanley (M. '39), State Highway Engr., State Highway Dept., Columbia, S.C.

Woolley, Ralf Rumel (M. '39), Senior Hydr. Engr., U. S. Geological Survey, 303 Federal Bldg., Salt Lake City, Utah.

Zehnder, Jack Eslie (Jun. '38), Junior Engr., U. S. Engr. Dept., 741 South Figueroa, Los Angeles (Res., 12123 Wilshire Boulevard, West Los Angeles), Calif.

Membership Transfers

Chase, John Howe (Assoc. M. '27; M. '39), Cons. Engr., 4323 Lemon St., Riverside, Calif.

Christopher, Willis Clinton (Assoc. M. '29; M. '39), Cons. Engr., Comisión Nacional de Irrigación, Balderas 94, Mexico, D.F., Mexico.

Epps, George Lasley (Jun. '29; Assoc. M. '39), Senior Engr., Design Dept., State Highway Comm., 10th and Van Buren St. (Res., 2515 Maryland St.), Topeka, Kans.

Fortson, Eugene Palmer, Jr. (Jun. '35; Assoc. M. '39), Asst. Engr., U. S. Waterways Experiment Station, Box 80 (Res., 1427 Chambers St.), Vicksburg, Miss.

Karakiz, Socrates Michael (Jun. '28; Assoc. M. '39), Engr., Commonwealth Edison Co. (Res., 3500 Lake Shore Drive), Chicago, Ill.

Levy, Jacob Herman (Jun. '27; Assoc. M. '39), Engr. and Contr., 5613 Pentridge St., Philadelphia, Pa.

Quirk, William Henry (Jun. '35; Assoc. M. '39), Topographical Draftsman, New York County Register, 31 Chambers St., Room 104, New York, N.Y.

Snyder, Howard Halsey (Assoc. M. '22; M. '39), Cons. Engr. (Ball & Snyder), 110 West 40th St., New York, N.Y.

Twinem, Joseph Conrad (Jun. '30; Assoc. M. '39), Operator, Civ. and Min Engr., and Geologist, Geneva-Matoa Consolidated Gold Mines and Associated Claims (Res., 313 Prospect St.), Cripple Creek, Colo.

Weir, Paul (Jun. '37; Assoc. M. '39), Supt. of Water Purification and Chf. Chemist, in Chg., Water Purification Plants (Res., 1194 Hemphill Ave., N.W.), Atlanta, Ga.

Reinstatements

Buente, Willard Harrison, M., reinstated April 4, 1939.

Estrada, Alfred Albert, Assoc. M., reinstated March 20, 1939.

Sacra, Charles, Assoc. M., reinstated March 15, 1939.

Sullivan, Jacob Butler, M., reinstated March 15, 1939.

Thomson, William Chase, M., reinstated March 23, 1939.

Resignations

Baldridge, Wilson Dwight, Jun., resigned March 14, 1939.

Lambert, William Braund, Jun., resigned April 6, 1939.

Wier, Robert John, Jun., resigned March 16, 1939.

Applications for Admission or Transfer

Condensed Records to Facilitate Comment from Members to Board of Direction

May 1, 1939 — NUMBER 5

The Constitution provides that the Board of Direction shall elect or reject all applicants for admission or for transfer. In order to determine justly the eligibility of each candidate, the Board must depend largely upon the membership for information.

Every member is urged, therefore, to scan carefully the list of candidates published each month in CIVIL ENGINEERING *and to furnish the Board with data which may aid in determining the eligibility of any applicant.*

It is especially urged that a definite recommendation as to the proper grading be given in each case, inasmuch as the grading must be based upon the opinions of those who know the applicant personally as well as upon the nature and extent of his professional experience. Any facts derogatory to the personal character or professional reputation of an applicant should be promptly communicated to the Board.

Communications relating to applicants are considered strictly confidential.

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of 30 days, and from non-residents of North America until the expiration of 90 days from the date of this list.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible Charge of Work
Member	Qualified to design as well as to direct important work	35 years	12 years	5 years RCM*
Associate Member	Qualified to direct work	27 years	8 years	1 year RCA*
Junior	Qualified for sub-professional work	20 years	4 years	
Affiliate	Qualified by scientific acquirements or practical experience to cooperate with engineers	35 years	12 years	5 years RCM*

* In the following list RCA (responsible charge—Associate Member standard) denotes years of responsible charge of work as principal or subordinate, and RCM (responsible charge—Member standard) denotes years of responsible charge of IMPORTANT work, i. e., work of considerable magnitude or considerable complexity.

FOR MEMBER

ADAMS, THOMAS CALDWELL (Assoc. M.), Salt Lake City, Utah. (Age 37) (Claims RCA 5.2 RCM 7.7) Oct. 1927 to June 1931 Asst. Prof., and July 1931 to date Associate Prof., of Civ. Eng., Univ. of Utah; since Oct. 1927 also Cons. Engr.

ARRASTIA, JUSTO, Pasay, Rizal, Philippine Islands. (Age 43) (Claims RCA 2.9 RCM 12.5) Sept. 1922 to Sept. 1926 and Dec. 1935 to date with Univ. of the Philippines as Instructor, Asst. Prof., and (since Dec. 1935) Lecturer; since Sept. 1926 Engr. and Contr.

BARRETT, LEIGH BLANTON, Atlanta, Ga. (Age 39) (Claims RC 15.4 D 4.8) Aug. 1935 to date with WPA, Albany and Atlanta Dists. as Supervisor of Operations, Director, and (since Aug. 1937) Asst. State Director of Operations, being Asst. to Director of Operations responsible for all WPA construction in Georgia; previously with FERA; Butler (Ga.) Naval Stores; U. S. Forest Service, in charge of design, etc.

BURRELL, GENE NATHANIEL, Knoxville, Tenn. (Age 49) (Claims RCA 11.6 RCM 7.0) Oct. 1933 to date Hydrographic and Hydr. Engr., TVA, in responsible charge of all silt investigations, including supervision of silt laboratory at Norris, etc.

COONS, HARRY CARMAN, East Lansing, Mich. (Age 53) (Claims RC 12.6 D 7.9) July 1933 to date with Michigan State Highway Dept. as Deputy Commr. and (except July 1937–Dec. 1938) Chf. Engr., having administrative supervision of design, construction, and maintenance of roads and bridges on state and federal system, also of research and testing div.

CRUISE, JOHN DONALD, East Lansing, Mich. (Age 42) (Claims RCA 5.5 RCM 9.8) Jan. 1924 to date with Michigan State Highway Dept. as Asst. Supt., Materials Engr., Office Engr., and (since Nov. 1935) Programming Engr., Highway Planning Survey.

EDLUND, LAWRENCE LINNE (Assoc. M.), Chicago, Ill. (Age 47) (Claims RC 18.2 D 17.8) Aug. 1928 to date with Armour and Co., Union Stock Yards as Superv. Engr., and (since Oct. 1931) Engr. of Design, with supervision over all building and mechanical design and specifications, special investigations, including reports and valuations.

EMERSON, LEWIS AZRO, Columbia, S.C. (Age 54) (Claims RCA 5.8 RCM 24.7) April 1939 to date Prin. Civ. Engr., under Chf. Engr., South Carolina Public Service Authority, Charleston, S.C.; 1938 to 1939 Director, South Carolina State Planning Board; 1936 to 1938 with National Resources Comm. as State Consultant for South Carolina (loaned by PWA); previously Engr.-Examiner, PWA, examining and reporting on engineering and financial feasibility of about 75 projects, etc.

FERRY, ARLOW VERNON, Halstead, Kans. (Age 41) (Claims RCA 4.1 RCM 10.1) Sept. 1936 to date Chf. Engr.; Black & Veatch, Cons. Engr.; previously Project Engr., Jackson County Highway Dept.; Dist. Engr., Missouri Relief and Reconstruction Comm.; Topographer, U. S. Geological Survey.

HANAVAN, WILLIAM LAWRENCE (Assoc. M.), New York City. (Age 56) (Claims RCA 12.5 RCM 13.0) 1906 to 1913 and 1938 to date with Board of Water Supply as Asst. Engr. and since 1938 Asst. Engr. (Designer); 1919 to 1937 Engr., Vulcanite Portland Cement Co., New York City.

HANSEN, VIGGO, Upper Darby, Pa. (Age 49) (Claims RCA 11.8 RCM 11.8) Dec. 1936 to date Civ. Structural Engr., Mech. Eng. Div., The Philadelphia Elec. Co.; previously Associate Engr., Procurement Div., Treasury Dept.; Concrete Engr., Du Pont Co.; Structural Engr. with J. T. Windrim, Archt., Philadelphia.

HENRY, CARLYLE FRANCIS, Washington, D.C. (Age 41) (Claims RCA 7.3 RCM 6.0) April 1935 to date Engr. Reviewer of Investigations, PWA, being Engr. Special Agent, making engineering investigations, reports, and recommendations on construction projects throughout United States; previously Chf. Planning Engr., Los Angeles County RA, being Head of Planning Dept.

JAMESON, WILLIAM HOWE (Assoc. M.), Bethlehem, Pa. (Age 36) (Claims RCA 5.4 RCM 6.2) March 1931 to date with Bridge Squad, McClintic-Marshall Corporation and Bethlehem Steel Co. as Squad Leader, and (since Dec. 1938) also Asst. Engr.

KOEBIG, ADOLF HEINRICH, JR. (Assoc. M.), Los Angeles, Calif. (Age 52) (Claims RCA 19.9 RCM 5.4) 1910 to date member of firm, Koebig & Koebig, Cons. Engrs.

LARGE, GEORGE ELWYN (Assoc. M.), Columbus, Ohio. (Age 39) (Claims RCA 6.0 RCM 8.8) Oct. 1929 to date with Ohio State Univ. as Instructor, Asst. Prof. of Civ. Eng., Senior Research Engr., and (since July 1934) Associate Prof., Dept. of Civ. Eng.

LESTER, HERBERT HAMILTON, Dayton, Ohio. (Age 47) (Claims RCA 6.3 RCM 11.8) July 1933 to date with CCC and Soil Conservation Service, U. S. Dept. of Agriculture, engaged successively as Camp Supt., Eng. Inspector, Superv. Engr., Project Engr., Project Mgr., and (since Oct. 1938) Regional Engr. on engineering for soil conservation, water storage, flood control, drainage, land utilization, etc.

MARK, JACOB, Brooklyn, N.Y. (Age 53) (Claims RCA 3.0 RCM 29.0) 1925 to date (until 1928 part of time) designing and building fireproof garages, tall structural steel framed, and reinforced concrete apartment houses, etc.

MOHR, HENRY ARTHUR, (Assoc. M.), Waban, Mass. (Age 53) (Claims RCA 10.8 RCM 18.2) Jan. 1918 to date with Raymond Concrete Pile Co., as Job Supt. controlling construction operations, and (since Jan. 1921) Dist. Mgr.

POE, HARRY TINKER (Assoc. M.), Charleston, S.C. (Age 56) (Claims RCA 0.6 RCM 28.1) Sept. 1934 to date with PWA as Engr. Inspector, State Traveling Engr. Inspector for South Carolina, Eng. Examiner for State Engr., Project Engr., Office Engr., Asst. Director, Inspection Div., Senior Engr., Eng. Div., and (since Nov. 1938) Prin. Engr., Santee-Cooper Power and Navigation Project; previously Appraiser, Home Owners Loan Corporation, Western Dist., S.C.

REEDER, KENNETH ABRAHAM, Los Angeles, Calif. (Age 51) (Claims RCA 12.4 RCM 7.0) Feb. 1919 to date with Southern California Edison Co., Ltd., as Draftsman, Designer, Supervisor, and (since 1932) Structural Engr., in charge of structural design, etc.

RIDGEWAY, GEORGE ALLEN (Assoc. M.), Jefferson City, Mo. (Age 54) (Claims RCA 7.4 RCM 19.3) Feb. 1922 to date with Missouri State Highway Dept. as Project Engr., Chf. of survey party, Supervising Maintenance Engr., First Asst. to Div. Engr., and (since Aug. 1931) Gen. Inspector of Constr.

RUNYON, FREDERICK OSCAR, Newark, N.J. (Age 64) (Claims RC 43.0) 1904 to date member of firm, Runyon & Carey, Cons. Engrs.

SCHADICK, CAMPBELL FREDERICK, Westport, New Zealand. (Age 40) (Claims RC 16.5 D 16.5) Jan. 1921 to date with Buller County Council as Engr. and Engr.-Secy., in charge of construction and reconstruction work, reporting on foundations, etc.

STEPHENS, UEL (Assoc. M.), Fort Worth, Tex. (Age 46) (Claims RCA 6.7 RCM 14.8) Nov. 1937 to date Associate Regional Engr., Region 5, PWA, assisting in supervising activities of 120 office engineers and 400 field engineers in servicing construction of 1,200 projects; March 1934 to Oct. 1937 Chf. Engr., Texas State Office, PWA; previously Engr. Examiner, Tex. PWA, reviewing and reporting on construction projects.

TOMPSON, WILLIAM RANDALL, Belmont, Mass. (Age 55) (Claims RCM 20.7) March 1926 to date Supt. of Streets, Supt. of Sewers, and Tree Warden.

FOR ASSOCIATE MEMBER

AMES, FRANK CLEMENT, Chattanooga, Tenn. (Age 31) (Claims RCA 4.9 RCM 0.0) Dec. 1929 to date with U. S. Geological Survey, Surface Water Div., Water Resources Branch as Jun. Hydr. Engr., and (since May 1934) Asst. Hydr. Engr.